THE UNIVERSITY OF ALBERTA

A STUDY OF TRAFFIC OPERATIONS AT RAMPS ON SELECTED ALBERTA INTERCHANGES USING PHOTOGRAPHIC TECHNIQUES

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance a thesis entitled A STUDY OF TRAFFIC OPERATIONS AT RAMPS ON SELECTED ALBERTA INTERCHANGES USING PHOTOGRAPHIC TECHNIQUES, submitted by DENNIS RAY DANCHUK in partial fulfilment of the requirements for the degree of Master of Science.

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ABSTRACT

An observation of traffic operations on two rural interchanges and one urban interchange in Alberta is made. The purpose of the project is to investigate the vehicle speeds and lane placement of the traffic, as well as the weaving maneuvers, accident records and geometric design of the rural study ramps, to gain a better understanding of the merging and diverging ability of Alberta drivers and to obtain an explanation of the single-vehicle accidents occurring at the rural interchanges.

The rural interchanges are of a cloverleaf design and the urban interchange of a diamond design.

The investigation employed a photographic technique. A 16 mm time-lapse camera at a camera speed of one frame per second was used to photograph the traffic movements and a time-motion analyzing projector was employed to analyze the film. A "giraffe" provided the necessary height for an aerial view of the rural study ramps; while a merge maneuver on the urban interchange was filmed from a nearby overpass.

The photographic method of traffic analysis is concluded to be a useful technique; but it is recommended that analysis of traffic be restricted to a distance of 600 feet from the camera.

It is concluded that drivers disregarded exit speed

signs and travelled the exit ramp at speeds greater than those recommended. Drivers aimed for the exit ramp curve and travelled on the right shoulder on the ramp, taking the shortest path around the curve. In addition, it is also concluded that the deceleration lane lengths studied were too small to permit a comfortable rate of deceleration, especially when exit was made on the inner loop. It is concluded that the single-vehicle accidents were the result of a high exit speed and either the lack of a spiral curve or a large degree of curvature. It is concluded that the two rural interchange ramps studied were designed at and below the minimum design recommendations of AASHO (1965).

It is concluded that the merge of the urban interchange operated well because of driver experience. Drivers entered the through lanes within 200 feet of the nose terminal and did not make full use of the acceleration lane.

It is recommended that a lesser degree of curve be employed in the design of future rural cloverleaf interchanges and that more diamond-type interchanges be constructed in rural areas. The inclusion of spiral curves in all exit ramps is also recommended. The trial use of rumble strips and safety grooving is also recommended. In addition, based on the operation of the urban merge maneuver analyzed, it is recommended that, where possible, rural interchanges in Alberta be designed to accommodate merge operations.

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CHAPTER I

INTRODUCTION

The intersection of two or more roadways at grade is of major importance in highway engineering and constitutes a commonplace, and often troublesome, aspect of roadway design. Generally, vehicles can pass through an intersection with a minimum of delay, especially when assisted by traffic control devices such as channelization (i.e. the separation of vehicles into separate paths through the use of pavement markings or raised islands) or signalization. However, whenever through and turning volumes become so great that traffic control devices are no longer adequate, a solution is usually sought by highway design engineers through the use of interchanges. An interchange permits vehicles to cross and enter intersecting roadways by the use of one or more grade separations (i.e. a structure which vertically separates two or more intersecting roadways). It also enables the highway design engineer to construct highways with control of access (i.e. access to a highway is fully or partially controlled by public authority).

However, the interchange has constantly been an accident prone location. The accident rate at

interchanges is roughly twice that between interchanges (Loutzenheiser, 1969).[a] Some of the more common accidents include: drivers unfamiliar with interchange operation have exited at high rates of speed and, unable to negotiate the curve, have left the roadway; and drivers have proceeded the wrong way on interchange ramps and collided head-on with an on-coming vehicle. As a result of accidents of this type at interchanges, the value of property damage has continued to soar.

It is this problem of traffic operations on interchanges which has prompted numerous highway design agencies to investigate the causes behind the high rate of accidents at interchanges. As a result of these studies, numerous design improvements have been implemented at rural and urban interchanges. Improved design of the entrance and exit ramp terminals, an improved curvature permitting higher exit speeds and a more natural driving path and the improved design of acceleration and deceleration lanes permitting a better interchange operation are only some of the results of investigations of interchange traffic operation.

It is the purpose of this study to investigate selected traffic operations on two rural cloverleaf-type interchanges in Alberta to document possible reasons behind the number of single-vehicle accidents occurring on these interchanges. The effects of the

[a]See list of references for all publications in brackets.

geometric design of these interchanges on vehicle speeds, lane placement, weaving maneuvers and accident records are considered in detail. At one location, the traffic operations on an outer connection, that is the ramp used for right turn movements, is considered; at the other location, traffic operations on an inner loop for left turn movements is analyzed. In addition, in order to obtain some information on the current merging ability of Alberta drivers, the vehicle speed and lane placement of vehicles operating on a diamond-type interchange is considered.

The observation of traffic operations employed a photographic technique in which a 16 mm time-lapse camera and a time-motion analyzing projector were used. A truck mounted elevating platform, commonly known as a "giraffe", provided the elevation necessary for an aerial view of the rural study ramps. An overpass near the urban interchange permitted an adequate view of the merge maneuver.

The study was financed and commissioned by the Planning Branch of the Alberta Department of Highways and Transport, and administered within the Alberta Cooperative Highway Programme by the Alberta Research Council, for the purpose of reporting on the traffic operations on two rural interchanges of typical design in Alberta.

Since the research was limited by time, finances and manpower, research methods were developed by the author to permit the investigation to remain within the restraints. Photographic and analyzing equipment were obtained from the University of Alberta and the author performed the photography and film analysis by himself. The study was initiated in May, 1970 with the photography and analysis portion of the study taking approximately three months to complete.

In subsequent chapters, a review of previous work of a similar nature is discussed. The methods and conclusions of other authors is presented. The theory relevant to interchange ramp design is discussed and the geometrics of the two rural study ramps are compared to recommended design specifications (i.e. AASHO (1965)). A complete description of the test locations, test apparatus and filming and analysis procedures of this study are provided in separate chapters. The results of the research are discussed in detail, with graphs and tables providing a visual interpretation of the results. The conclusions and recommendations of the author comprise the final two chapters. Appendices provide a more detailed explanation of the test locations, equipment, an example of the computer programs and the rural study ramp geometrics.

CHAPTER II

PREVIOUS TRAFFIC OPERATIONS ANALYSES

This chapter is a review of those articles which had the greatest influence on the procedures employed in this investigation. All studies employed a photographic technique and, although their objectives may have been different from those of this investigation, their data collection procedures provided a basis for techniques used during this research. The major items dealt with in this chapter include filming and analysis procedures and procedures for determining the speed and lane placement of vehicles.

Davis and Williams (1968) studied ". . . vehicle operating characteristics such as speeds, headways, lateral placements and decelerations, to evaluate the operating efficiency of existing deceleration lanes and suggest changes in design if warranted."

Keese, Pinnell and McCasland (1960) purported ". . . to determine the effects of certain geometric features of freeways on traffic operation through a study of the actual operation of vehicles on representative sections of freeways in Texas cities."

Newman (1963) investigated weaving maneuvers and the influence on weaving maneuvers of lane

re-stripping.

Berry, Ross and Pfefer (1963) purported " . . . to examine the present status of knowledge about left-hand ramps, to study the operational patterns of traffic at some high-volume left-hand exit ramps in Illinois and to draw conclusions about the suitability of such ramps."

Only the Davis and Williams (1968) report had an objective similar to that of this study.

Each report is reviewed according to techniques for data collection, speed determination and lane placement. Analyzing equipment and conclusions are also considered under separate headings.

2.1 Data Collection

Davis and Williams (1968) had their films taken from the top of a 50 foot standard construction scaffolding which had been guyed to reduce windsway. A 16 mm movie camera, battery powered and with a variable frame speed, was employed. They considered a camera speed of 8 frames per second would give the required accuracy for measuring vehicle speeds, headways and placements. Seven day traffic counts of through and turning traffic were obtained using two automatic traffic counters at preselected locations prior to the commencement of filming. Filming was then performed

during summer daytime peak hours on dry roads with the highest combination of through and turning traffic for approximately thirty minutes. The tower was occupied for a week prior to filming in order that motorists become accustomed to it.

Keese, Pinnell and McCasland (1960) employed three types of towers for filming. First, a tower truck parked on an overpass structure, then a 48 foot temporary tower and, finally, a 60 foot portable tower. A 16 mm motion picture camera was used with filming performed at both 8 and 10 frames per second. They believed these camera speeds allowed the accurate determination of vehicle speeds, headways and other desirable traffic characteristics. In addition, a 12 inch electric clock with a sweep second hand was positioned to appear in an unused portion of each frame of the motion picture film. The tower was erected and occupied for one week prior to filming. It is pointed out that neither the tower nor the personnel on the tower influenced the pattern of the freeway traffic. Continuous motion pictures were taken of each test section for approximately one hour and thirty minutes during the morning and evening peak periods and for one hour during off-peak conditions.

Newman (1963) employed a 16 mm time-lapse movie camera with a camera speed of one frame per second.

The camera was mounted on the directional sign preceding the on-ramp nose and operated from the ground.

Berry, Ross and Pfefer (1963) employed two and frequently three or more 16 mm movie cameras driven by " . . . synchronous electric motors at either 60 or 100 frames per minute, powered either from a 110-V power supply or from a power pack consisting of a 12-V battery and an invertor." Overpasses provided the usual vantage points for filming, although the apparatus was sometimes clamped to utility poles. The camera speed was checked by placing a colored filter in front of the lens at specific intervals of time. This study was interested in distribution of speeds by lane, the point of exit onto the deceleration lane and the frequency of occurrence of hazardous maneuvers.

2.2 Speed Determination

Davis and Williams (1968) employed target boards and spots of white paint spaced at 40 foot intervals on the straight portion of the deceleration lane and at 20 foot intervals along the curved portion of the deceleration lane and the exit loop. This was done because the authors were concerned that white lines painted across the pavement shoulders and lanes would influence driver behavior. The target boards were filmed at the reference points during the first portion of each

film. When the film was analyzed the film was advanced until the image of the first target boards appeared. The outlines of the pavement were then sketched onto the screen and the location of all the grid lines, as indicated by the target boards, were plotted. In this manner, a grid was produced and, knowing the distance between grid lines and the vehicle travel time between grid lines by a frame count, vehicle speeds were determined. The distance from the film channel of the projector to the screen was recorded in order that the distance could be duplicated if the equipment was accidently moved.

Keese, Pinnell and McCasland (1960) employed transverse white lines painted on the pavement 176 feet apart to provide reference points for speed determination. Vehicle speeds were determined by observing each vehicle and recording a frame count as the rear wheel of the vehicle crossed a transverse white line painted on the pavement. A second frame count was recorded for the same vehicle as its rear wheel crossed a second line 176 feet in advance of the first line. Because the camera was operated at a constant speed, it was possible to determine the time required for the vehicle to travel the 176 feet and to compute the speed of the vehicles in miles per hour.

Newman (1963) determined speed by superimposing

a grid on the film, and recording the distance travelled over a given number of frames.

Berry, Ross and Pfefer (1963) used whitewash lines on the shoulder of the road spaced at 50 or 100 foot intervals to supply reference points for speed determination. The film was advanced a frame at a time with a frame counter providing the time base. Then, knowing the distance between reference lines and the vehicle travel time between them, vehicle speed was determined.

2.3 Lane Placement

Davis and Williams (1968) determined lateral position by dividing the roadway longitudinally into five equal segments by proportion and recording the location of the right front tire of the vehicle within one of the five equal longitudinal segments.

Keese, Pinnell and McCasland (1960) determined lane placement by recording the distance from an outside curb to either the left or right rear wheel of the vehicle with a special scale when the vehicle tire was directly over the transverse line on the pavement 176 feet in advance of the entrance ramp.

2.4 Analyzing Equipment

Davis and Williams (1968) employed a 16 mm

time-motion study projector with a single frame advancement and a frame counter. The film image was projected onto a table top screen which consisted of ink tracing paper taped to a drawing board. This set-up permitted grid lines and pavement edges to be drawn on the screen. Film warping from the heat of the projector lamp was noted as a problem during intensive study of any one frame of film.

Keese, Pinnell and McCasland (1960) employed two projectors to analyze their motion pictures. The first maintained constant focus and contained a daylight screen in the machine, and was used to obtain time-space relationships. This projector was especially constructed so that it could be stopped for "still" or single-frame viewing. Because the film was held firmly between two glass plates, warping from lantern heat was eliminated. A microfilm reader was used to obtain placement data. It projected an image at a fixed magnification set-up which could be scaled. Both projectors provided the advantage of being able to replay or re-create traffic situations as well as stopping the film to permit more comprehensive visual analysis of each frame of the movie.

Newman (1963) employed a projector which could be completely controlled by the operator for speed of projection or advanced or reversed frame by frame. Various projection speeds were possible with this apparatus.

Berry, Ross and Pfefer (1963) employed commercial movie projectors which back projected the images onto ground glass screens. A parallactic grid was constructed on this screen from the shoulder markings, thus giving a distance base. The projector had provision for advancing the film one frame at a time.

2.5 Conclusions

Davis and Williams (1968) concluded deceleration lanes were not used as intended. Drivers did not pull over far enough to the right when they started decelerating and aimed for the curve at the end of the deceleration lane. Drivers used the right 15 feet of pavement on the loops and followed the straightest path around single curves. Exits with the least amount of curvature satisfied motorists best.

Keese, Pinnell and McCasland (1960) concluded freeway volume control is possible and increases the operating efficiency of the freeway. In addition, lanes carry approximately the same percentage of traffic during the peak hour and drivers tend to reference their driving to their left side.

Newman (1963) concluded re-stripping of lanes in the weaving area provides a smoother merge operation with higher merging speeds and improvement in the use of the weaving lane.

Berry, Ross and Pfefer (1963) concluded left-hand exits do not hamper proper freeway operation but adequate signing is required in advance of the left-hand exit ramp to inform drivers that the exit ramp is on the left-hand side.

In summary, it may be stated that numerous approaches to gathering information from films of traffic movements have been employed, e.g. white reference lines, target boards. However, the use of time-lapse motion picture equipment and projectors with an attachment permitting single frame advancement has been a standard procedure in most traffic analysis studies employing photographic equipment.

CHAPTER III

INTERCHANGE DESIGN and OPERATIONS

The design of an interchange involves the application of numerous principles of highway engineering. This chapter discusses the more important aspects of interchange design with primary consideration given to the design of interchange ramps.

3.1 General

The Highway Capacity Manual (1965) defines an interchange as " . . . a system of interconnecting roadways in conjunction with one or more grade separations, providing for the interchange of traffic between two or more roadways or highways on different levels." In this study, emphasis was placed on the traffic movements on and geometric design of inner loops and outer connections which together may be classified as ramps. The Highway Capacity Manual (1965) defines the terms ramp, inner loop and outer connection thusly:

> "Ramp - An interconnecting roadway of a traffic interchange, or any connection between highways at different levels, or between parallel highways, on which vehicles may enter or leave a designated roadway.

Inner loop - A ramp used by traffic destined for a left-turn movement from one of the through roadways to a second when such movement is accomplished by making a right-exit turn followed by a three-quarter-round right-turn maneuver and a right entrance turn.

Outer connection - A ramp used by traffic destined for a right-turn movement from one of the through roadways separated by a structure to the second through roadway."

With the exception of the definition of a truck, definitions of other highway engineering terms used throughout this thesis are analagous to those in Chapter Two of the Highway Capacity Manual (1965). For the purpose of this thesis, a truck was considered as any vehicle having one or more axles with dual tires. This definition would, therefore, include vehicles commonly classified as buses.

The interchange is a facility employed by highway design engineers to promote the free flow of through traffic at the intersection of two major highways. As is the case with all techniques available in highway engineering, the interchange does have its advantages and disadvantages. The AASHO publication "A Policy on Geometric Design of Rural Highways - 1965" summarizes the good and bad features of interchanges very nicely. It cites the chief advantages as:

"1. The capacity of the through travelled ways within the interchange can be made to approach or equal that outside the interchange.

2. Increased safety is provided for through and left turning traffic. Right turning movements make the same maneuver as on at-grade intersections but generally on a much higher type facility which also results in greater safety.

3. Stops and appreciable speed changes are eliminated for through movements.

4. The highway grade separation is flexible in design and may be adapted to almost all likely angles and positions of intersecting roads.

5. Interchanges are usually adaptable to stage construction.

6. The grade separation is an essential part of the highest type of highway - the expressway or freeway."

The AASHO publication further cites the major disadvantages of interchanges stem from economic and topographic factors. It states the chief disadvantages are:

"1. Highway grade separations and interchanges

are costly.

2. Interchanges are not foolproof as regards traffic operation. The layout may be confusing (particularly where there is not a complete complement of ramps) to some drivers, especially the unfamiliar. However as driver experience with interchanges has increased, better usage has resulted.
3. The undercrossing grade separation does not lend itself to stage construction.
4. A grade separation may make it necessary to introduce undesirable crests and sags in the profile of one or both intersecting highways, particularly in flat topography.
5. A simple type interchange is not readily adaptable to a multileg intersection with five or more approaches."

3.2 Interchange Ramps

The "Traffic Engineering Handbook (1965)" published by the Institute of Traffic Engineers defines the basic design criteria for interchange ramps as:

"1. Design speed
2. Minimum radii
3. Superelevation rates
4. Superelevation run-off rate

5. Compound curves and transitions
6. Sight distance
7. Grades
8. Vertical curves
9. Cross section
10. Deceleration and acceleration lanes
11. Terminal design."

Space does not permit a detailed discussion of each point. Therefore, each point is considered briefly in the following paragraphs and the reader is referred to design manuals such as the CGRA design publication entitled "Manual of Geometric Design Standards for Canadian Roads and Streets (1967)", the AASHO publication "A Policy on Geometric Design of Rural Highways - 1965" or the Institute of Traffic Engineers publication "Traffic Engineers Handbook (1965)" which contain a more detailed consideration of the design criteria.

Design Speed

The ramp design speed is usually controlled by economic and topographic factors. However, as high a design speed as practicable is the recommendation offered by all highway design manuals. Since the design speed for loops is often near the minimum recommended value, adequate speed change lanes must be incorporated into the overall ramp design. The same holds true for the design of outer connections with a minimum design speed.

"Table IX-2" in "A Policy on Geometric Design of Rural Highways - 1965" lists the highway design speeds and corresponding ramp design speeds and minimum curve radii for a safe ramp design.

Minimum Radii

The radius of curvature of a ramp is related to numerous factors including the design speed, superelevation, side friction, presence or absence of deceleration or acceleration lanes and transition curves. The radius of a curvature should be as large as is feasible within the restraints of economy and topography. The minimum radius is determined from the centrifugal force formula:

$$R = \frac{V^2}{15(e+f)}$$

where: R = the radius of the horizontal curve in feet

V = the assumed design speed in miles per hour

e = the value of maximum superelevation in feet per foot of width

f = the maximum side friction value employed.

The values of radii for varying values of superelevation, e, and side friction, f, are listed in "Tables 17.14 and 17.15" in the "Traffic Engineering Handbook (1965)."

Superelevation Rates

The values of superelevation, e, as recommended in the "Manual of Geometric Design Standards for Canadian Roads and Streets (1967)" published by the Canadian Good Roads Association, varies between 0.02 - 0.08 feet per foot, with 0.06 feet per foot a maximum value where icing is prevalent, as in Alberta.

Superelevation Run-off Rate

The "Traffic Engineering Handbook (1965)" states in "Table 17.34" that, as the design speed increases, the change in superelevation rate per station decreases, i.e. from 0.08 feet per foot at 15-20 mph to 0.05 feet per foot at 35 or more mph.

Compound Curves and Transitions

Compound curves and transitions provide the natural travel path which most drivers follow while turning at interchanges. "A Policy on Geometric Design of Rural Highways - 1965" states that " . . . provision for natural travel paths is best affected by the use of transition or spiral curves, which may be inserted between a tangent and a circular arc or between two circular arcs of different radii." The lengths of spiral curves for various turning design speeds are contained in "Table VIII-4" in the same publication with a recommended minimum spiral length of 100 feet.

Insofar as compound curves are concerned, their chief advantage is their use in introducing desirable shapes of ramps at interchanges. The aforementioned publication also states that " . . . when circular arcs of widely different radii are joined, the alignment appears abrupt or forced and the travel paths of vehicles operating thereon do not closely follow the roadway or else considerable steering effort is required. General observations on ramps having differences in radii with a ratio of 2 indicate that both operation and appearance normally are satisfactory." However, " . . . where feasible, a lesser difference in radii should be used; a desirable maximum ratio is 1.75."

Sight Distance

The main sight distance requirement in ramp design is provision for a safe stopping sight distance. It is the sum of the distance travelled during perception and brake reaction time and the distance travelled while braking to a stop. The formula for computing safe stopping sight distance is:

$$SSD = 1.47PV + \frac{V^2}{30(f \pm g)}$$

where: SSD = safe stopping sight distance in feet

V = speed from which stop is made in miles per hour

P = perception-reaction time (usually 2.5 seconds)

f = coefficient of friction

g = percent of grade divided by 100 (added for upgrade and subtracted for downgrade).

Suggested values for stopping sight distances are included in "Table III-1" in "A Policy on Geometric Design of Rural Highways - 1965."

Grades

Maximum grades are usually controlled by their influence on truck speeds and the presence or absence of snow. Recommended grade values range between 3-8% with a maximum of 12% allowed in mountainous terrain, on interchange ramps and in some urban areas. A maximum of 5% is recommended in areas where snow cover is prevalent. The relation between design speed and maximum grade is shown in "Table 17.12" in the "Traffic Engineering Handbook (1965)."

Vertical Curves

The "Traffic Engineering Handbook (1965)" states " . . . the minimum lengths of vertical curves, except where grades are excessive such as on steep ramps, are usually governed by either safe-stopping or safe-passing sight distance in the case of crest curves, and headlight sight distance in the case of sag curves." The minimum length of vertical curve for design may be computed from

the formula:

$$L = KA$$

where: L = length of vertical curve in feet

K = constant related to sight lines and geometry of a parabolic curve (see "Table 17.11, Traffic Engineering Handbook (1965)")

A = algebraic difference in grades in percent.

Cross Section

The type of pavement, be it high, intermediate or low, usually controls the cross slope. The high type pavements retain their shape and do not ravel at the edges. Loss of width or unequal settlement are not allowed for when the pavement is placed on a stable subgrade. Furthermore, the smoothness of high type pavements offers little frictional resistance to the flow of surface water and enables drivers to steer easily and keep their vehicles in proper paths. Consequently, it becomes feasible to design high type pavements with minimum cross slopes. Low type pavements, which are usually loose, must be crowned enough to drain well so as to avoid softening of the surface. Furthermore, they tend to ravel at the edges, reducing the effective width and requiring greater steering effort to maintain a correct path. The normal pavement cross slopes

are included in "Table IV-1" in "A Policy on Geometric Design of Rural Highways (1965)." They range from 0.01-0.02 feet per foot for high type pavements to 0.02-0.04 feet per foot for low type pavements.

The recommended lane width is 10-12 feet with shoulder widths varying from 4-10 feet depending on the type of highway facility.

It is recommended that highways with two or more lanes in each direction have a separating median. Median widths vary between 4-70 feet, with a 25 foot median providing an elimination of traffic noise, driving tension and headlight glare. A median width of at least 50 feet is recommended for high-speed expressways.

Deceleration and Acceleration Lanes

Both acceleration and deceleration lanes can be generally classified as speed change lanes. "A Policy on Geometric Design of Rural Highways (1965)" defines a speed change lane as " . . . an auxilary lane, including tapered areas primarily for the acceleration or deceleration of vehicles entering or leaving the through traffic lanes." A speed change lane need not be of uniform width but should be wide and long enough to enable a driver to maneuver his vehicle onto it properly and once on it, to make the necessary change between the speed of operation of the highway and the lower speed on the turning roadway. Recommended design lengths of deceleration and acceleration

lanes, including the taper, are set forth in "Table VII-10" in "A Policy on Geometric Design of Rural Highways (1965)." A more complete discussion of this aspect of ramp design is also included in Chapter VII in the same publication.

Terminal Design

The "Traffic Engineering Handbook (1965)" states that " . . . exit ramp terminals should be designed so that (1) there is a clear indication to the driver that this is a point of departure from the through lanes, and (2) the act of leaving the through lanes can be accomplished without slowing down, (the deceleration being accomplished on the turning roadway or on a parallel deceleration lane after leaving the through traffic lanes)." A nose offset from 4-12 feet is recommended with greater offsets provided as the turning roadway becomes more important. The length of nose taper on the through lane side should be sufficient to permit a through driver, who has erroneously deviated to the right, to clear the nose and return to the through pavement beyond it without encroaching on the shoulder. Values for the minimum length of taper beyond an offset nose range from 7-16 feet depending upon the speed of the approach highway. The range of values for varying design approach speeds are included in "Table VII-16" in "A Policy on Geometric Design of Rural Highways (1965)." Sight distance should also be considered in exit terminal

design. Sight distance should be adequate so that drivers would be able to observe any delay on the ramp well in advance without their visibility obscured by bridges and embankments. Minimum sight distances are included in "Table 17.9" in the "Traffic Engineering Handbook (1965)."

Each of the aforementioned points has been considered in the comparison between recommended ramp design and the "as constructed" ramps at the Salisbury and Bremner interchanges in Chapter VII of this thesis.

3.3 Recent Design Proposals

In a report by Loutzenheiser (1969), it was stated that, based on past experience, the operation of interchanges has not been satisfactory. The accident rate at interchanges is roughly twice that of the roadway between interchanges. With this in mind, the American Association of State Highway Officials has published reports with design recommendations to improve operations and safety. These include: "Report by the Special Freeway Study and Analysis Committee to the Executive Committee of the AASHO" February 1960, and "Highway Design and Operational Practices Related to Highway Safety," February 1967.

For ramp design, new recommendations include the provision of only one exit terminal from the main lanes for each interchange crossroad. It should be located on the right and upstream from the crossroad bridge.

Also in the report by Loutzenheiser (1969), it was stated that " . . . parallel deceleration lanes (exit ramp) were not used by drivers following in the reverse curve path of the edge. Drivers exited from the main roadway by a single curve path following a small deviation angle away from the main roadway towards the exit roadway. Based on this, many States have adopted the four or five degree straight edge tapered deceleration lane. Where space permits, this tapered alinement is continued past the physical approach nose or gore. Alinement through and partially past the gore area should be very good so as to provide drivers a chance to orient themselves to the more restricted driving environment on the ramp and the crossroad. The horizontal and vertical alinement on the ramp proper is also being made a higher order than initially. Curvature is being flattened, reverse curvature is being reduced but more important sight distance is being improved to eliminate the hidden dip or the unseen sharp curve."

In addition, concern has been expressed on the number of fixed objects adjacent to ramps, especially those in the gore area, since objects in this area are difficult to protect.

In conclusion, the North America design of highways and interchanges has been progressively changed over the past ten years. Increasing traffic volumes on rural highways coupled with higher accident rates at interchanges had led to this constant reappraisal of design standards.

CHAPTER IV

TEST LOCATIONS

There were two primary filming locations, namely the Salisbury interchange and the Bremner interchange, both east of Edmonton. A third filming location at Groat Road and 107 Avenue within Edmonton city limits was chosen after it was realized that there were few merging maneuvers in the rural areas chosen. This chapter describes briefly the test locations and the reasons behind their selection as study sites.

4.1 Salisbury and Bremner Interchanges

The traffic operations on a ramp and deceleration lane were studied at both the Salisbury and Bremner interchanges. There were three main reasons for the selection of these interchanges as study sites:

1. They represented a fairly standard interchange design employed throughout the province.
2. They were close to the city permitting relatively easy access for research purposes.
3. They had a single-vehicle accident rate which required some investigation.

The Salisbury interchange is located on the Sherwood Park Freeway roughly 3 miles east of the Edmonton

city limits at the intersection with Highway Number 14X. The outer connection in the northeast quadrant formed the study area at this interchange. The Bremner interchange is situated on Highway Number 16 roughly 8 miles east of the Edmonton city limits at the intersection with Highway Number 55, north of the community of Fort Saskatchewan. The inner loop of the southeast quadrant formed the study area at the Bremner interchange. Both interchanges are of typical cloverleaf design and both of these locations are shown with respect ot Edmonton in FIGURE IV.1.

4.2 Groat Road and 107 Avenue

While filming the diverge maneuvers and during traffic counts at the Salisbury and Bremner location, it was noted that there was never sufficient volume to produce separate, simultaneous traffic streams on either the through lanes or the ramps to permit a merging maneuver. Furthermore, it was primarily on the diverge portion of the selected ramps that the single-vehicle accidents were occurring. It was therefore decided that a merging maneuver situation would be filmed within the Edmonton city limits to investigate the merging ability of some Alberta drivers. Consequently, the interchange at Groat Road and 107 Avenue was chosen. Film was taken of traffic turning off 107 Avenue and travelling south on Groat Road. This location had several advantages:

1. During the morning peak, a large volume of

FIGURE IV.1 RURAL STUDY LOCATIONS

traffic used this particular merge location.

2. An overpass close to the interchange provided an excellent vantage point for filming:

   i. It provided an aerial view of the complete ramp and merge area.

   ii. The photographic equipment remained inconspicuous to unsuspecting drivers.

The location of this diamond-type interchange within Edmonton is shown in FIGURE IV.2.

A more complete description of all test locations is included in APPENDIX A.

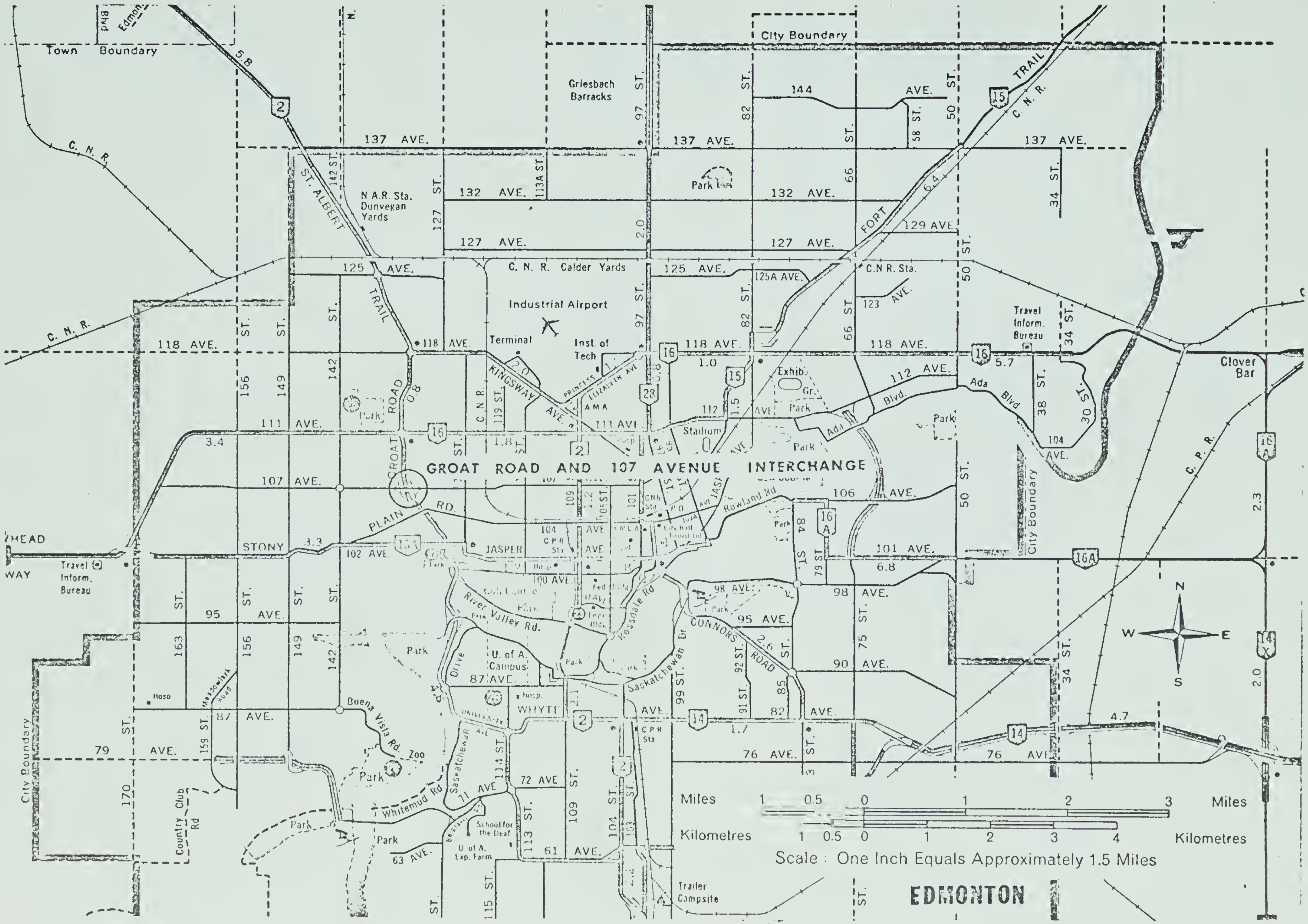

FIGURE IV.2 URBAN STUDY LOCATION

CHAPTER V

TEST APPARATUS

This chapter briefly describes the apparatus employed in both the photography and the analysis of the film during the investigation. A more detailed description of the equipment is included in APPENDIX B. In addition, a description of the ball bank indicator and its use is included at the end of this chapter.

5.1 Camera and Accessories

At the commencement of the investigation, it was decided that some arrangement involving a time-lapse camera would provide the best form of photographic technique. This decision was reached, in part, through a review of the literature published by other highway departments which had performed similar studies. The advantage of reviewing traffic movements at will, when such information as speed, lane placement and volume could be obtained, was also considered. In addition, the time-lapse feature of the equipment would allow a certain interval of real time (i.e. one hour) to be condensed into a three minute run of the 100 foot reel of film, depending on the frame interval, e.g. a camera speed of one frame per second condenses one hour of filming time into three minutes of viewing time

at 16 frames per second.

The equipment was obtained from the Motion Pictures branch of the Technical Services Department of the University of Alberta.

The apparatus consisted of a camera, tripod, cycle timer and power supply regulator. However, because of the necessity for a 110 volt AC power supply, an inverter and a 12 volt car battery were added to the standard equipment. With a fully charged battery, a filming time of one hour was easily attained. In addition, although the cycle timer provided a reliable means of setting the frame interval, a clock with a sweep second hand was set into the field of view of the camera so that a check of the interval could be easily made. This equipment, excluding the clock, is shown in FIGURE V.1. A detailed explanation of this apparatus can be found in APPENDIX B.

5.2 Field Filming Equipment

The major piece of equipment used in the field during the filming sessions was a "giraffe" obtained from the Alberta Department of Public Works. This vehicle was especially useful because it provided the necessary mobility for quickly moving from one location to another. Its boom, which extended to a height of 40 feet, provided an aerial view of the study ramps. Its bucket, which could support 900 pounds, was large enough to hold the operator

FIGURE V.1 CAMERA AND ACCESSORIES

and all the filming equipment. Furthermore, the boom and the bucket were stable under gusty wind conditions.

5.3 Film Analysis Equipment

A special analyst projector was obtained from the Chemical Engineering Department of the U. of A. This equipment was useful because in incorporated a screen placed on the top of a special cabinet type desk and a mirror. This allowed an analysis of all films in a seated position looking down on the screen. In addition, either single frame or any one of several multiple frame advancements provided a choice of film viewing speeds. No film warping from excessive lamp heat was encountered, even though individual frames were viewed for along periods of time. The complete analyzing apparatus, as set up for use, is shown in FIGURE V.2. A more complete description of the projector and its accessories can be found in APPENDIX B.

5.4 Ball Bank Indicator

The ball bank indicator is an instrument which can be used to determine advisory speed limits on horizontal curves. These advisory speed limits are generally not legally enforceable, but in some courts violation of the advisory speeds is admissible as evidence that the driver was operating in a reckless manner. Exit speed limits are an example of posted advisory speeds.

FIGURE V.2 ANALYZING EQUIPMENT

The relation between speed and the transverse coefficient of friction can be calculated using the formula:

$$V = \sqrt{\frac{(e+f)R}{0.067}}$$

where V = speed of vehicle in miles per hour

e = superelevation in feet per foot of horizontal width

f = transverse coefficient of friction

R = radius of curvature in feet.

In this study, this device was used to provide a rough estimate of the coefficient of side friction of the study ramps at the Salisbury and Bremner interchanges. It consists of an arc shaped instrument approximately four inches long with degree graduations extending from zero degrees in the middle to twenty degrees on both sides of the zero mark. A bubble in the device provides the means of determining the angle.

Its use is based on the principle that with such a device mounted in a vehicle in motion, the ball bank reading at any time is indicative of the combined effect of the body roll angle, the centrifugal force angle and the superelevation angle. Safe speeds on curves result in ball bank readings of 14 degrees for speeds below 20 mph, 12 degrees for speeds between 25-30 mph and 10 degrees for speeds of 35 mph and higher. The corresponding side friction

factors at these speeds are 0.21 for speeds below 20 mph, 0.18 for speeds between 25-30 mph and 0.15 for speeds of 35 mph or more (from AASHO (1965)). The results obtained with this device by the author are included in CHAPTER VII of this thesis.

This concludes the brief description of the apparatus used in all phases of the research. A detailed description of all test apparatus, excluding the "giraffe" and the ball bank indicator, is included in APPENDIX B

CHAPTER VI

TEST PROCEDURES

The test procedures used during this study were developed after a review of the literature. Several innovations were introduced by the author when it was discovered that methods of others would not apply to this investigation. This chapter describes the procedures employed during the photography and analysis of the film and the use of the ball bank indicator in determining side friction factors.

6.1 Preliminary Procedures

Filming was performed from predetermined locations at all interchanges. However, prior to any photography, manual traffic counts were taken for three mornings at both the Salisbury and Bremner interchanges to determine the peak hour. In both cases, this was found to be between 7-8 A.M. These counts were only based on traffic volumes for the ramps under study.

Because of the time assumed necessary for film analysis, it was decided that three films, 100 feet in length, would be taken at both the Salisbury and Bremner interchanges.

White lines roughly 3-4 feet long and four to

five inches wide were then painted on the right shoulder in the direction of travel of the study ramps at the Salisbury and Bremner interchanges. The lines were spaced at 50 foot intervals on both the ramp and the deceleration lane and were intended to provide reference lines for vehicle speed and lane placement determination.

It was also decided beforehand, that filming would be performed on dry weekdays only. Dry periods eliminated such variables as reduced side friction and reduced visibility from influencing driver behavior. The morning peak hour during weekdays provided the greatest volume on the study ramps.

6.2 Filming Procedures

6.2.1 Salisbury Interchange

Filming at the Salisbury interchange required the "giraffe" on all occasions and was performed first. The "giraffe" was parked on the service road to a subdivision abutting the Sherwood Park Freeway. This location provided an adequate view of the interchange and, in particular, the study ramp. Because few drivers acknowledged the author while he was filming and because the "giraffe" was adequately sheltered by tree cover paralleling the highway, the presence of the truck and the photographer did not influence driver behavior. No advance warning was given of the study prior to the actual filming. The vehicle was

just parked on the three mornings of filming and the film taken.

The view of the study ramp permitted analysis of speed and lane placement of diverging traffic on both the highway and the ramp. The weaving maneuvers and volume of diverging traffic was also readily available from the film with this view.

Upon review of the film for this location, it was discovered that the white lines painted on the shoulder of the pavement were not visible. Therefore, the distance between numerous lamp and power poles paralleling the roadway was obtained. These poles provided reference points on the film which were readily usable for determining the required information.

6.2.2 Bremner Interchange

Filming was then performed for four mornings at the Bremner interchange. A procedure similar to that followed at Salisbury was employed. The "giraffe" was parked in the ditch area parallel to the through lanes of Highway Number 16 on the south side of the highway downstream of the study ramp. The complete diverge maneuver was photographed for three mornings. In addition, because the vehicle was far enough downstream of the ramp, driver behavior was not influenced.

The white lines on the pavement shoulder were not visible at this location either and, consequently, points

of merging and diverging pavements, signs, abutments and the end of the guardrail on the study ramp were used as reference points on the films. Information on vehicle speed, lane placement, volumes and weaving maneuvers was obtained.

In addition, the filming equipment was set up for one morning on the northeast embankment of the overpass. This filming was done without the "giraffe". The ramp was photographed from this position primarily to obtain data on the lateral position of vehicles as they travelled the study ramp. This information was not available from the previous filming location. The white lines on the shoulder were used at this location as reference points for lateral position and vehicle speed determination.

6.2.3 Groat Road and 107 Avenue

Only one film was taken at this location because of the high volume of traffic and the time required to analyze the vehicle movements. The 7-8 A.M. peak was filmed. The filming equipment was set up on the Stony Plain Road overpass downstream from the merge area. This particular overpass was stable and vibration of the filming equipment did not occur, as it did when a test film was taken from the Bremner overpass. Lamp poles paralleling the right hand side of the merge lane and Groat Road were employed as reference points for the vehicle speed and lateral position determination.

6.3 Film Analysis

The film analysis involved the use of the equipment described in both Chapter V and APPENDIX B of this thesis. The analysis procedure was developed by the author, although the basic concept is similar to that employed by Davis and Williams (1968).

The analysis of all films was performed in the same manner. A sheet of transparent paper was overlaid onto the screen and, with the film held at one frame, chosen at random near the beginning of the film for its clarity, the perimeter of the roadway was traced onto the paper. The poles and other objects which were used as reference points were also traced onto the film. The paper was then removed and the distances measured in the field between the reference points were placed on the paper in equal increments, i.e. 5 foot intervals. Objects on the other side of the highway directly across from the reference points were used to draw lines perpendicular to the highway. The paper was then placed, in a loose condition so it could be moved, on the screen and the film advanced until a diverging vehicle was found. With the perimeter of the highway and the reference points "lined-up" on the film and the tracing paper, the distance travelled per second, by either the front or rear right wheel, across the reference lines was recorded. Because the camera speed was one frame per second, the distance travelled by a

vehicle through two adjacent frames provided the velocity in feet per second. This distance per second was later converted to miles per hour and provided the velocity data.

The vehicle lane placement was determined in a similar manner. The pavement edge was traced onto a sheet of transparent paper and, knowing the pavement width at the reference points, the pavement was divided into equal segments i.e. 5 feet, and lines parallel to the pavement edge were traced onto the sheet. The film was then advanced until a diverging vehicle was obtained. The lateral position of the front or rear right wheel across each of the reference lines was then recorded.

In this manner, velocity and lateral position of vehicles at numerous positions on the deceleration lane and study ramp was obtained.

The volume was obtained by setting the equipment on "auto" at one frame per second and counting the vehicles which used the study ramp or the through lanes.

Trucks were analyzed and counted separately from cars to provide a percentage of trucks and to provide some information on the difference, if any, between the operating characteristics of cars and trucks on the study ramp.

It should be mentioned here that, although the camera lens used for filming was in focus for any object from three feet to infinity in front of it, those vehicles which were analyzed at 1500 feet from the camera, although

being in focus, were difficult to analyze because of their small size in proportion to the overall frame size.

As can be appreciated, the film analysis was an extremely tedious procedure and a substantial amount of time was required for the analysis of all films.

6.4 Computer Programs

After the velocity and lane placement data was obtained, a computer program was written which arranged the numbers in descending order of magnitude, determined the percentile (i.e. percent less than or equal to) and determined the arithmetic mean and standard deviation of the values obtained at the various positions for all three study locations.

In addition, another computer program was written which determined the average rates of deceleration and acceleration for cars and trucks from the arithmetic mean of the speeds at all three locations.

An example of these two programs is contained in APPENDIX C.

6.5 Ball Bank Indicator

The ball bank indicator was used to provide a rough estimate of the side friction values of the study ramps at the Salisbury and Bremner interchanges. The instrument was supported in a level position on the opened lid of the

glove compartment of the test vehicle by an assistant to the author. The vehicle was then driven around the test ramps at varying rates of speed and the corresponding angle was recorded. A description of the ball bank indicator is provided in Chapter V and the results obtained with it by the author are included in Chapter VII.

This concludes the description of the test procedures employed during the data gathering portion of the research. The filming procedure involved, in most cases, the use of a "giraffe" to gain an aerial view of the study ramp. The analyzing procedure was an empirical but tedious operation incorporating a grid system to determine vehicle speeds and lane placement. The procedure followed to determine side friction factors involved the use of a ball bank indicator. The analysis portion of the research was the most time consuming of all the test procedures.

CHAPTER VII

RESULTS

This chapter presents the volumes, speeds and lateral position of vehicles at each of the study locations. The weaving maneuvers, geometric design and accident records of the Salisbury and Bremner study ramps are also presented.

7.1 General

The techniques employed to obtain the results were those of CHAPTER VI. The figures showing the speed and lane placement are the presentation of the output of a computer program shown in APPENDIX C, FIGURE C.1.

The velocity distribution figures are plotted with the "PERCENT OF VEHICLES LESS THAN OR EQUAL TO" on the vertical axis and the "VELOCITY" in miles per hour on the horizontal axis. They represent typical cumulative frequency plots, with "best fit" curves shown for each reference position at the three study locations. The 85th percentile values are shown because they include most of the drivers and usually form the basis for advisory speed limits. The 50th percentile value or average value is also shown. This value was readily available from the computer program shown in APPENDIX C. FIGURE C.1. Because "best fit" curves were drawn through the points on the graphs, the 50th percentile

and the average value do not necessarily correspond. Therefore, the discussion of average values uses the arithmetic mean values of the computer program, since these can be considered more correct. Car and truck velocity distributions are presented on separate graphs for all three study locations.

The car lane placement figures are presented as histograms showing the placement of the right front or rear wheel from the right pavement edge. The placement results at all reference positions are shown for the three study areas. Truck lane placement is not presented as a histogram because of an insufficient number of vehicles to provide reliable results. Average values of car and truck lane placement for each study location are provided in tables and discussed throughout the presentation of results.

Average deceleration rates are presented to provide information on the rate of change of car and truck velocities as the vehicles exit from the through lanes. This information is useful in determining whether or not deceleration rates are uncomfortable to driver and passengers and if deceleration lane lengths are adequate.

The deceleration rates were computed using the formula:

$$v_o^2 - v_i^2 = 2as$$

where: v_o = final velocity (ft/sec)

v_i = initial velocity (ft/sec)

a = acceleration (ft/sec/sec)

(deceleration, if negative)

s = distance (ft).

The values for v_o and v_i were the average velocities of cars and trucks as they crossed the various reference points. The computer program used for the deceleration rate calculation is shown in APPENDIX C, FIGURE C.2.

The average velocity, lane placement and deceleration or acceleration values for each study interchange are provided in separate tables throughout the presentation of results. The standard deviation of the velocity and lane placement distributions are also provided. All these values were available from the computer programs described in APPENDIX C. Car and truck values are presented separately.

The arithmetic mean was determined using the formula:

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

where: $\bar{x}$ = arithmetic mean

x_i = individual measurement

n = total number of observations.

An estimate of the standard deviation was computed using the formula:

$$s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{(n-1)}}$$

where: s = estimate of the standard deviation

x_i = individual measurement

$\bar{x}$ = arithmetic mean

n = total number of observations.

The "F test" and the "t test" were used to determine the statistical significance of the velocity results. Car velocities were compared with truck velocities at the three study locations to determine if they were significantly different. Car and truck velocities of one location were not compared with those of another because of different speed limits and operating conditions.

The "F test" was employed to determine if the two variances compared were significantly different. The F value was calculated using the formula:

$$F = \frac{s_1^2}{s_2^2}$$

where: F = F value

s_1, s_2 = estimates of the population variances (i.e. estimate of the standard deviation of cars, 1, and trucks, 2) at a reference point, $s_1 > s_2$.

The calculated F value was compared to the tabulated F value at the 5 percent level of significance. On this basis, the null hypothesis (i.e. that there is no significant difference between the distributions being

compared or that they are drawn from the same population) was either accepted or rejected.

If the null hypothesis was accepted, the "t test" was performed at reference positions on or near the study ramp. The following formulae were employed during the "t test":

$$s_c^2 = \frac{s_1^2(n_1-1) + s_2^2(n_2-1)}{(n_1-1) + (n_2-1)}$$

where: s_c = combined variance

s_1, s_2 = estimate of the standard deviation of the samples (i.e. cars = 1, trucks = 2) at the reference points

n_1, n_2 = total number of cars, 1, and trucks, 2.

$$s_d = s_c \sqrt{\frac{n_1+n_2}{n_1 n_2}}$$

where: s_d = standard deviation of the difference of the means

s_c, n_1, n_2 = as above.

$$t = \frac{\bar{x}_1 - \bar{x}_2}{s_d}$$

where: t = t value

$\bar{x}_1, \bar{x}_2$ = sample means (i.e. mean velocity of cars and trucks) at reference positions,

$\bar{x}_1 > \bar{x}_2$.

s_d = standard deviation of the difference of the means.

The calculated t value was then compared to the tabulated t values at the 5 percent level of significance. If the calculated t was greater than the tabulated t value, the null hypothesis was rejected and it was concluded that the difference between car and truck velocities was significant.

The results obtained with the statistical analysis are discussed in this chapter. Calculations employed to obtain the results are presented in APPENDIX D.

7.2 Salisbury

7.2.1 Salisbury Volume

The total observed highway volume, over a three hour filming period, was 3670 vehicles. Of this total, 221 vehicles or 6% used the study ramp. Trucks accounted for 2% of the total traffic on the highway and 5% of the traffic on the study ramp. The distribution of traffic on the study ramp was 209 cars and 12 trucks.

7.2.2 Salisbury Velocity

The vehicle speeds were computed for diverging vehicles as they crossed nine reference points on the highway, deceleration lane and ramp. These points are shown in FIGURE VII.1 and in APPENDIX A, FIGURE A.1. Positions

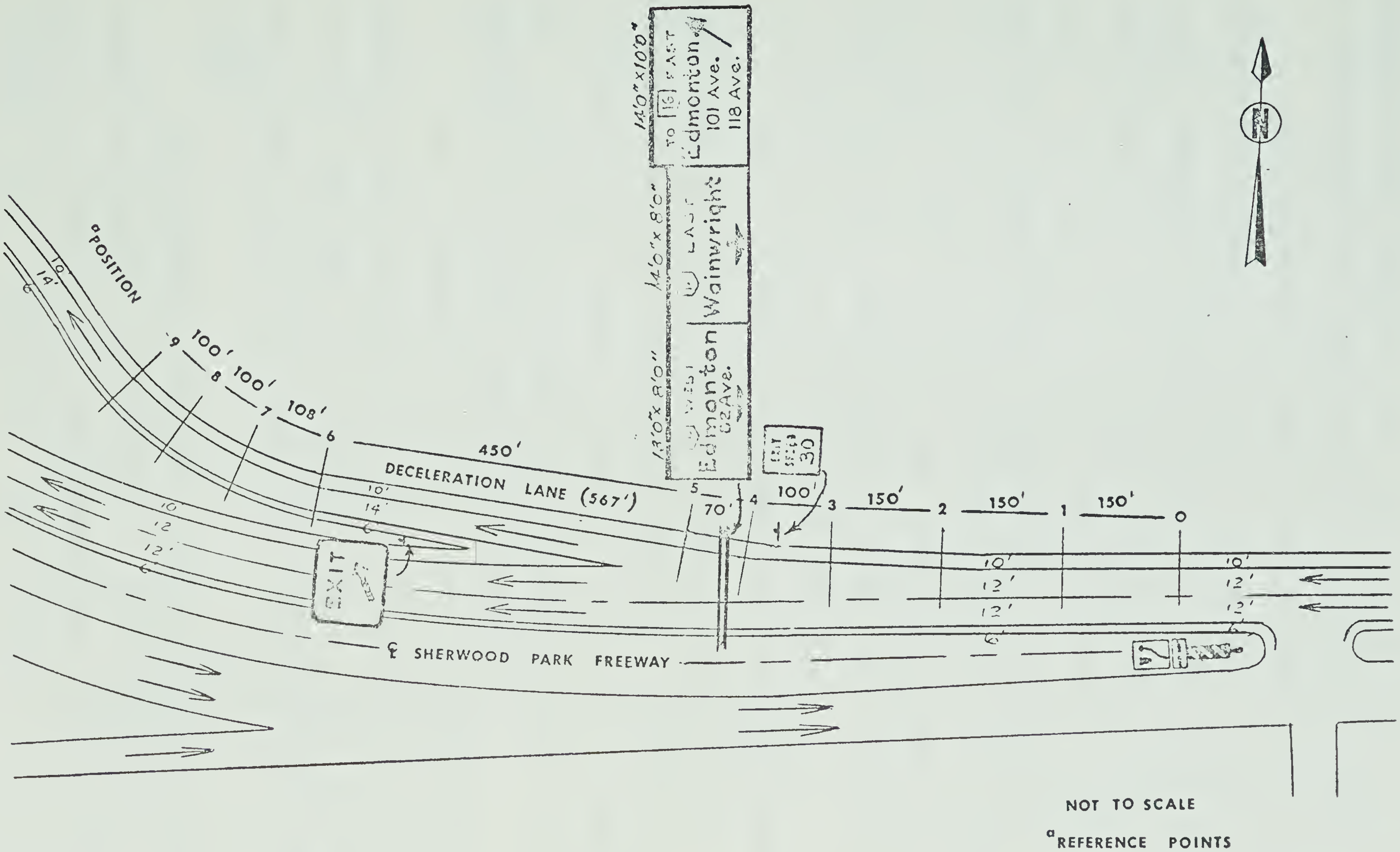

FIGURE VII.1 SALISBURY STUDY AREA

1-3 were on the highway, 4 and 5 were on the first part of the deceleration lane and 6-9 were located on the ramp.

The car velocity distribution is presented in FIGURE VII.2. Because velocities were similar across several reference points and because the scale of the graph would not permit a clear separate presentation of the results, the velocities across several reference points have been represented by a single line. The velocities across positions one and four were similar and are represented by a single line. Similarly, the velocities across positions two and three are represented by a single line, as are the car velocities across positions six and seven.

The 85 percentile car speed on the highway increased from 62 mph at position one to 64 mph at position three. At the beginning of the deceleration lane (i.e. position 4), the 85 percentile car velocity decreased to 62 mph. At position five, 70 feet further along the deceleration lane, the 85 percentile car velocity decreased to 58 mph.

The 85 percentile speed on the ramp varied from 39 mph at the ramp entrance (i.e. position six) to 46 mph past the first ramp curve (i.e. position nine). In no case was the car speed on the ramp less than 27 mph or greater than 56 mph.

The arithmetic mean and standard deviation of the speeds are presented in TABLE VII.1. The average car velocities at the first four positions were similar with

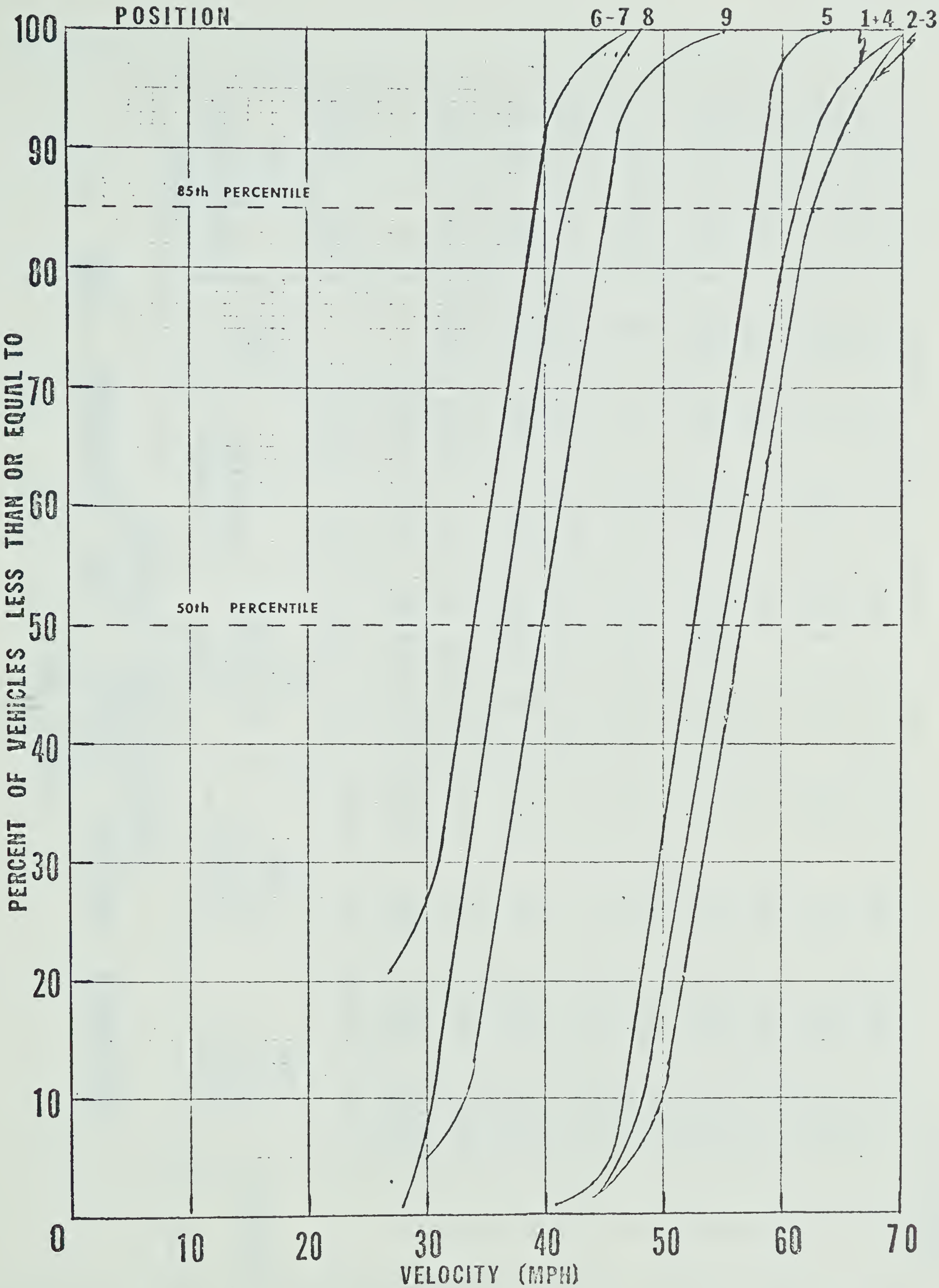

FIGURE VII.2 SALISBURY CAR VELOCITY DISTRIBUTION

TABLE VII.1

SALISBURY AVERAGE VELOCITY, LANE PLACEMENT AND DECELERATION VALUES

POSITION	AVERAGE VELOCITY (MPH)		STANDARD DEVIATION (MPH)		AVERAGE LANE PLACEMENT (FT)		STANDARD DEVIATION (FT)		DIST. (FT)	AVERAGE DECELERATION RATE (FPSPS)	
	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS		CARS	TRUCKS
1	55.83	49.67	5.66	6.30	12.85	13.00	1.34	1.21	—	—	—
2	57.45	51.25	5.38	6.55	12.59	12.75	1.11	0.97	150	-1.32	-1.14
3	57.43	51.75	5.51	6.38	12.35	12.67	0.91	0.98	150	0.02	-0.37
4	55.94	51.67	5.14	6.27	12.13	12.42	0.87	0.90	100	1.82	0.09
5	53.14	49.67	4.46	5.26	11.90	12.17	0.80	0.72	70	4.69	3.11
6	37.11	33.83	4.81	4.34	11.20	10.33	1.26	1.50	450	3.46	3.16
7	34.59	33.50	5.35	3.92	10.47	9.50	1.41	1.68	108	1.80	0.22
8	37.11	36.17	5.27	4.00	9.87	8.50	1.50	1.45	100	-1.94	-2.00
9	40.52	39.58	5.86	5.14	10.45	9.50	1.45	0.90	100	-2.85	-2.78

[a] TWO DECIMAL PLACES ARE NOT MEANT TO IMPLY DEGREE OF ACCURACY; ARE FOR PURPOSES OF STATISTICAL ANALYSIS ONLY

averages in the 55 mph range. As the vehicles entered the deceleration lane, the speed shows a slight reduction to an average of 53 mph. The average car velocity at the ramp entrance was 37 mph, which is 7 mph in excess of the posted exit speed. As the cars continued on the ramp, they decelerated until they passed the initial portion of the curve. A slight acceleration was then noted as they continued around and exited out of the first curve on the ramp.

From TABLE VII.1, the standard deviation of the average car velocity across all the reference points was approximately five miles per hour. This value appears to indicate most drivers travelled near the average velocity.

The average car deceleration rates, as shown in TABLE VII.1, can be considered small (i.e. the largest being 4.69 ft/sec^2), since the AASHO (1965) states that deceleration rates up to 9 ft/sec^2 are comfortable to driver and passengers.

The truck speeds at the various positions are shown in FIGURE VII.3. The velocity distribution across positions one to five were similar and are represented by a single line on the graph; as are the velocity distributions across positions six and seven. The speed pattern was similar to that of the cars although the speeds were lower for trucks.

The 85 percentile truck speed increased from 38 mph at the ramp entrance (i.e. position six) to 46 mph past

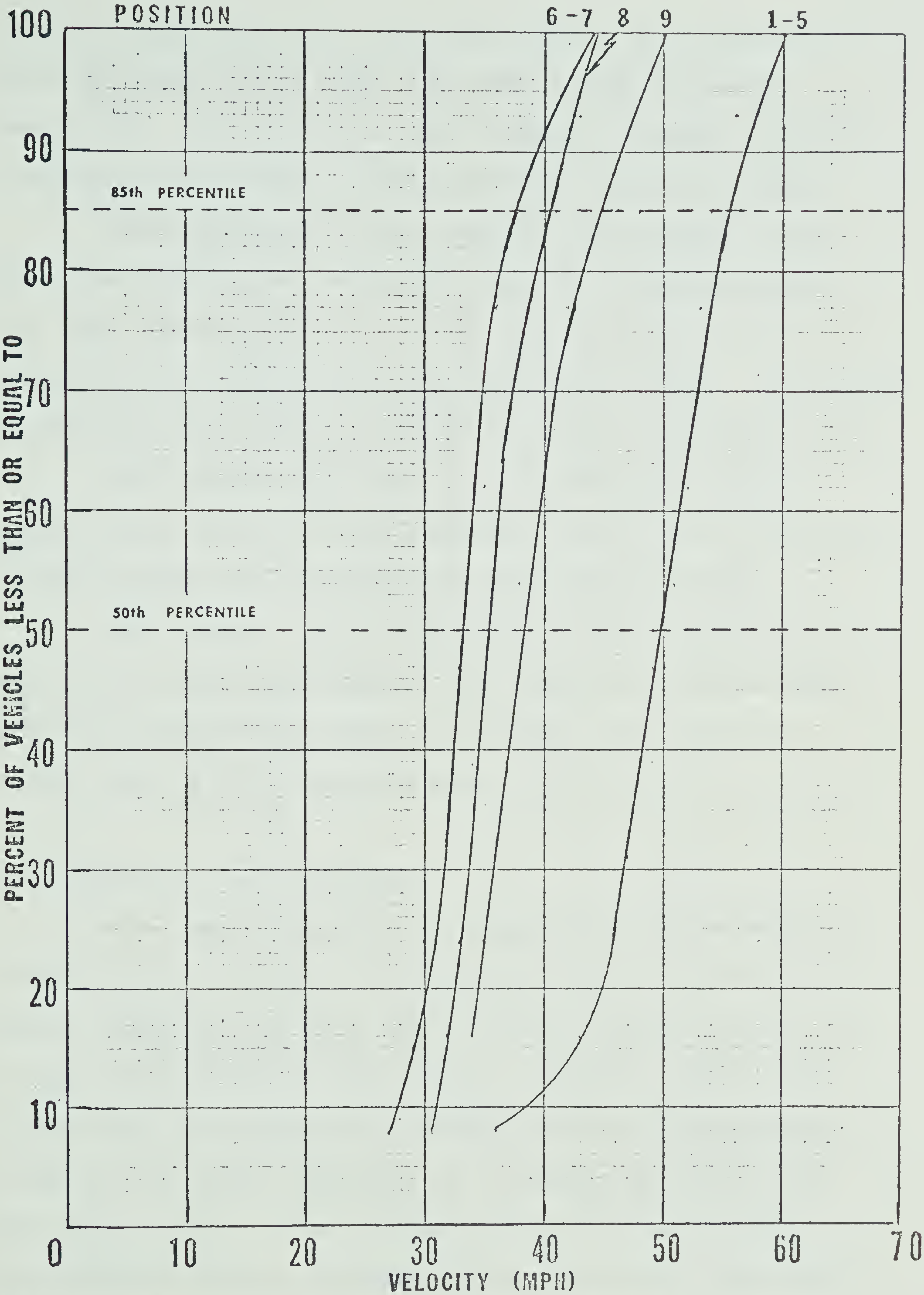

FIGURE VII.3 SALISBURY TRUCK VELOCITY DISTRIBUTION

the first ramp curve (i.e. position nine). In no case was the truck speed on the ramp less than 27 mph or greater than 50 mph. The lack of a large number of trucks resulted in velocity distribution curves that are almost straight.

From TABLE VII.1, the average truck speed was 51 mph on the highway and the first part of the deceleration lane (i.e. positions 1-5). At the ramp entrance (i.e. position six), the average truck speed was 34 mph. Thereafter, the average truck speed increased to 40 mph at position nine.

From TABLE VII.1, the standard deviation of the average truck velocity varied between 4 and 6 mph, indicating a similar speed was maintained by most truck drivers.

The average truck deceleration rates, as shown in TABLE VII.1, were more uniform than those of cars and were within the comfortable rate of 9 ft/sec^2. The greatest average truck deceleration rate was 3.16 ft/sec^2.

7.2.3 Salisbury Lane Placement

The lateral position of Salisbury cars is shown in FIGURES VII.4A and VII.4B. From positions one to five, the results indicate that less than 5% of the drivers travel on or near the shoulder of the highway. However, the results of positions six to nine were different with the percentage of shoulder driving increasing to a maximum of 66% on the first ramp curve. It was evident from the film analysis and results that drivers aimed for the ramp curve and took the shortest path around it. The higher the exit velocity,

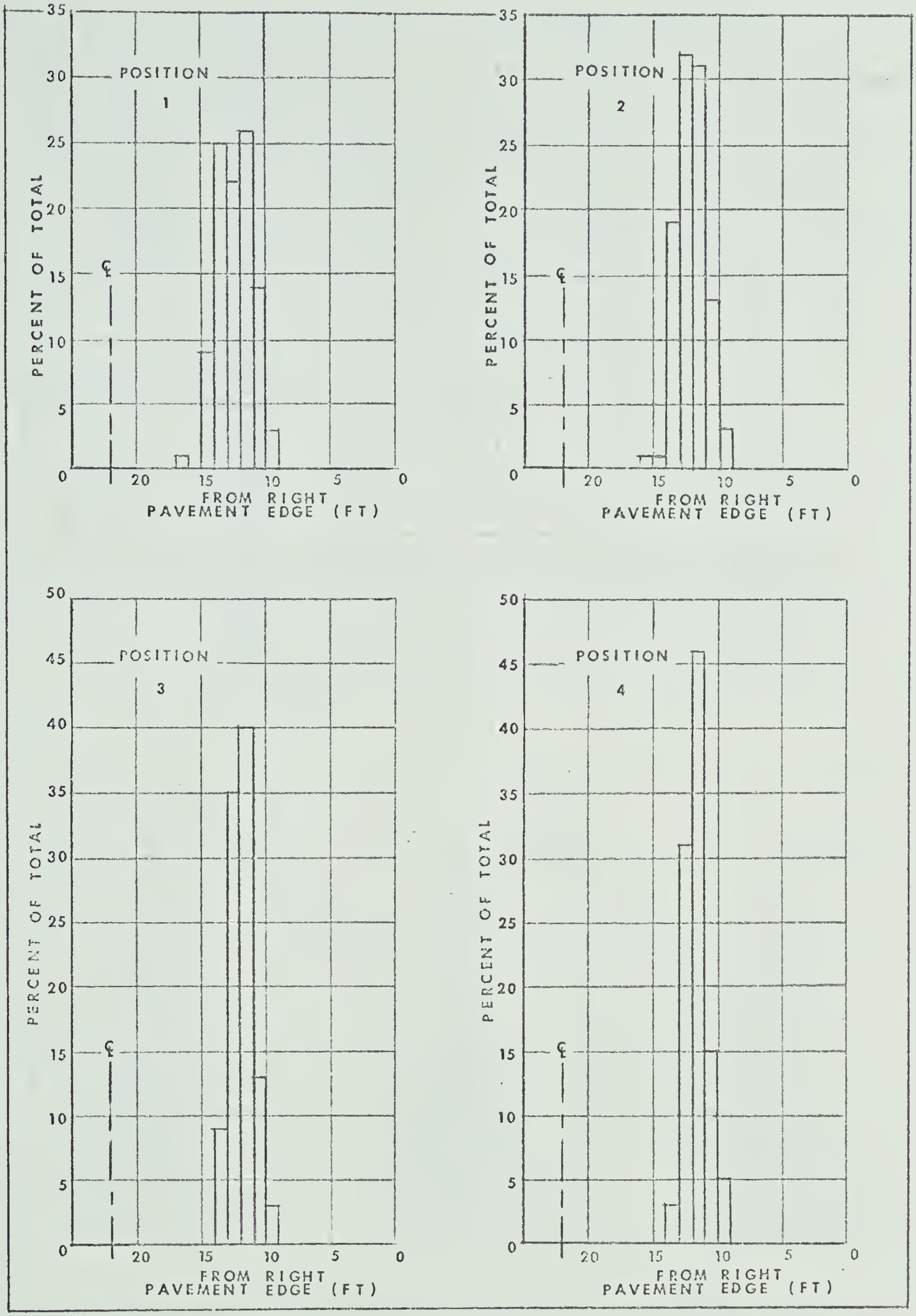

FIGURE VII.4A SALISBURY CAR LANE PLACEMENT

[a] 10 FOOT RIGHT SHOULDER EXISTS AT ALL POSITIONS

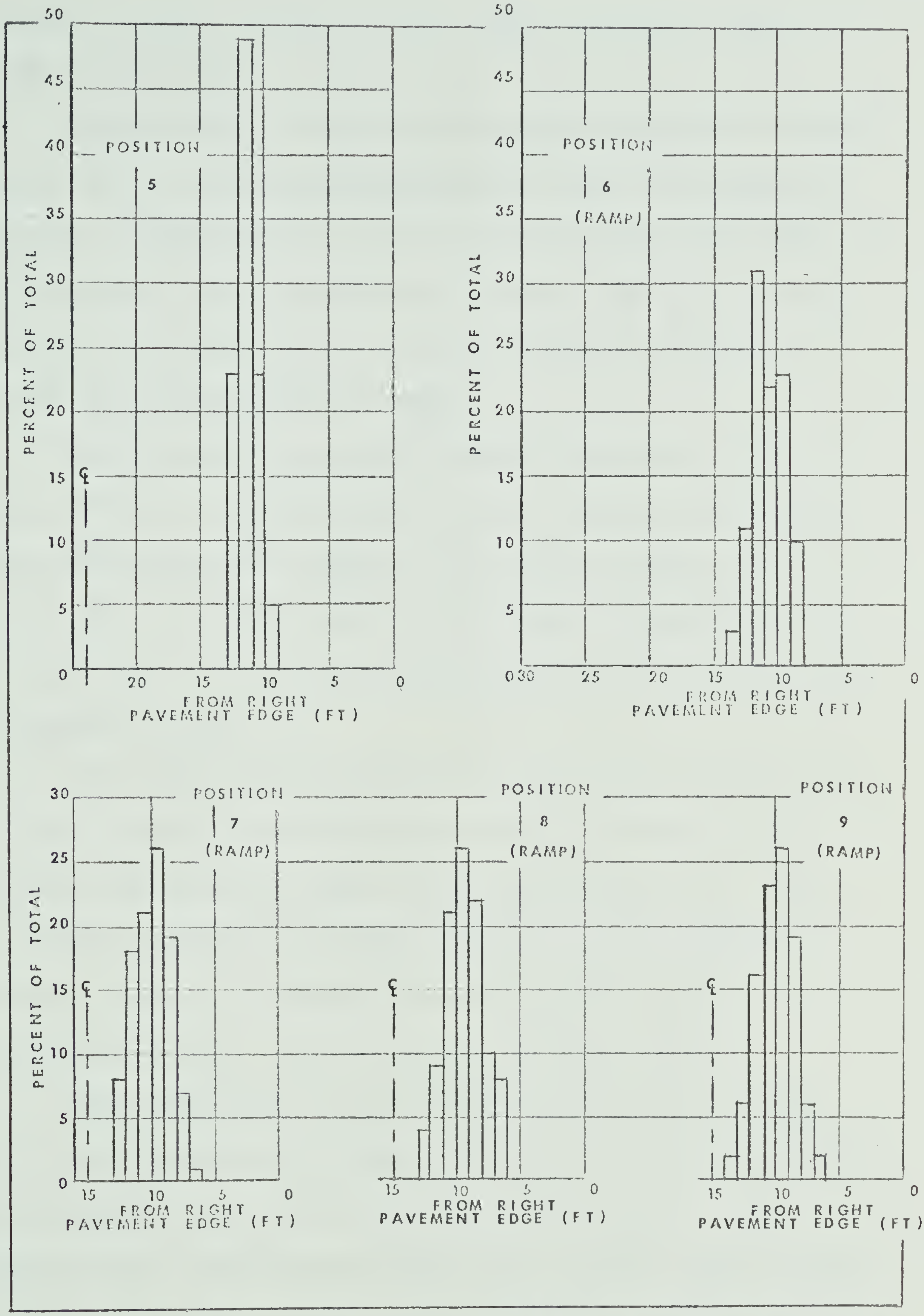

FIGURE VII.4B SALISBURY CAR LANE PLACEMENT

[a]10 FOOT RIGHT SHOULDER EXISTS AT ALL POSITIONS

the closer the vehicle was to the right pavement edge on the first ramp curve.

The arithmetic mean and standard deviation of lane placement at the various positions for cars and trucks is summarized in TABLE VII.1. It should be noted that there is a 10 foot paved right shoulder throughout the entire study area which is indicated by a yellow pavement marking 10 feet from the right pavement edge.

The average car lane placement decreased from roughly 13 feet at position one on the highway to 11 feet from the right pavement edge at the ramp entrance (i.e. position six). On the ramp, the average placement of the right rear wheel of cars was approximately 10 feet from the right pavement edge.

From TABLE VII.1, it can be seen that the average truck lane placement was similar to that of cars on the highway and deceleration lane (i.e. positions 1-5). However, shoulder driving was more evident on the ramp for trucks than cars (i.e. positions 6-9), with trucks approximately 9 feet from the right pavement edge.

From TABLE VII.1, the standard deviation of the average lane placement was approximately one foot for both cars and trucks across all the reference points. This indicates that, depending on their exit speed, most drivers travelled in similar paths on the deceleration lane and the ramp.

Generally, vehicles travelled with their right rear wheels 12 feet from the right pavement edge on the tapered deceleration lane. Neither cars nor trucks travelled the left 6 feet of the ramp, with only 3-6% of the traffic travelling with their right wheels 14 feet from the right pavement edge.

7.2.4 Salisbury Statistical Analysis of Velocities

Because velocities on the study ramp were of major concern, the "F test" and the "t test" were applied to car and truck velocities on the ramp only. The results of these tests indicate truck velocities were significantly different from car velocities only at the ramp entrance. At other positions along the ramp, significant differences between car and truck velocities would be due to chance alone. The results were determined at a 5 percent level of significance or a probability of 5 percent that any difference between car and truck velocities may occur by chance and not be due to real causes.

7.2.5 Salisbury Weaving Maneuvers

Only one vehicle of those studied did not travel in the deceleration lane before entering the ramp. This vehicle was travelling at a high rate of speed in the inside lane and cut in front of several vehicles when the exit maneuver was performed.

Drivers on the outside lane, with no intention of exiting, would change to the inside lane if an exiting vehicle was reducing speed on the outside lane. This maneuver was usually possible and the through vehicle did not become "trapped" behind the exiting vehicle. Through drivers would change to the inside lane up to 400 feet in advance of the nose terminal. Thereafter, exiting vehicles were sufficiently off the through lanes and on the deceleration lane to permit two lanes of vehicles to pass.

With less than 100 exiting vehicles per hour during the peak period of the day, weaving maneuvers were not sufficiently plentiful to be hazardous.

7.2.6 Salisbury Geometrics

A summary of the Salisbury interchange study ramp geometrics is shown in APPENDIX E, TABLES E.1A, E.1B and E.1C.

The relevant geometric features include a low ramp design speed, a radius of curvature that is too small for the speeds travelled by 80-90% of the ramp traffic (i.e. speeds greater than 35 mph) and the absence of a spiral curve.

The ball bank indicator readings obtained from trial runs on the study ramp are shown in TABLE VII.2. They indicate that exit speeds greater than 35 mph are unsafe since the ball bank angle reading increases drastically for every 10 mph increase in exit speed from 30-50 mph. The

TABLE VII.2

SALISBURY BALL BANK INDICATOR VALUES

SPEED MPH	BALL BANK RDG., DEGREES		CORRESPONDING [b] "f" VALUES	
	[a]RECMMD.	TEST	[a]RECMMD.	TEST
30	12	5 5	0.18	0.18
40	10	13 11	0.15	0.14
50	10	18 20	0.15	-

a RECOMMENDATIONS FROM AASHO (1965), p. 154.

b TRANSVERSE COEFFICIENT OF FRICTION.

lack of a spiral curve leading into the exit ramp appears to be the major cause, since the ball bank reading is an indication of the centrifugal force, body roll angle and the superelevation. As the exit speed increases, the centrifugal force and body roll angle increase more than would be the case if a spiral curve were provided. A spiral curve would provide a more natural driving path.

7.2.7 Salisbury Accident Record

Between 1966 and 1970 there were 10 accidents on the Salisbury study ramp. The first of these was a typical "wrong way" accident with drivers unfamiliar with the interchange. The remainder were single-vehicle accidents distributed as follows: five drivers who failed to negotiate the first curve into the ramp under dry conditions, two drivers who skidded out of control on either a wet or icy pavement and two drivers who were cut off. Only one driver was impaired.

The drivers who failed to negotiate the first ramp curve under dry conditions are of major concern in this study. It appears that the lack of a spiral curve and an exit speed probably greater than the posted limit of 30 mph caused these drivers to leave the roadway.

7.3 Bremner

7.3.1 Bremner Volume

The total highway volume observed over a four hour

filming period was 856 vehicles. Of this total, 540 vehicles or 63% used the study ramp. Trucks accounted for 12% of the total traffic on the highway and 7% of the traffic on the study ramp. The distribution of traffic on the study ramp was 500 cars and 40 trucks.

7.3.2 Bremner Velocity

The vehicle speeds were computed for diverging vehicles as they crossed seven reference points on the highway, deceleration lane and ramp, as shown in FIGURE VII.5 and in APPENDIX A, FIGURE A.2.

The car velocity distribution is presented in FIGURE VII.6. Positions 1-5 on the deceleration lane showed a marked reduction in car velocities with the 85 percentile speed changing from 59 mph at the start of the deceleration lane to 29 mph near the end of the deceleration lane. The 85 percentile speed of cars entering the ramp (i.e. position six) was 24 mph. The speed increased slightly to an 85 percentile value of 27 mph as drivers climbed the ramp grade which changes from two to four percent between positions six and seven, a distance of 160 feet.

The arithmetic mean and standard deviation of the speeds are presented in TABLE VII.3. The average car velocities decreased from 56 mph at the start of the deceleration lane to 25 mph near the end of the deceleration lane. The average car velocities on the ramp increased from

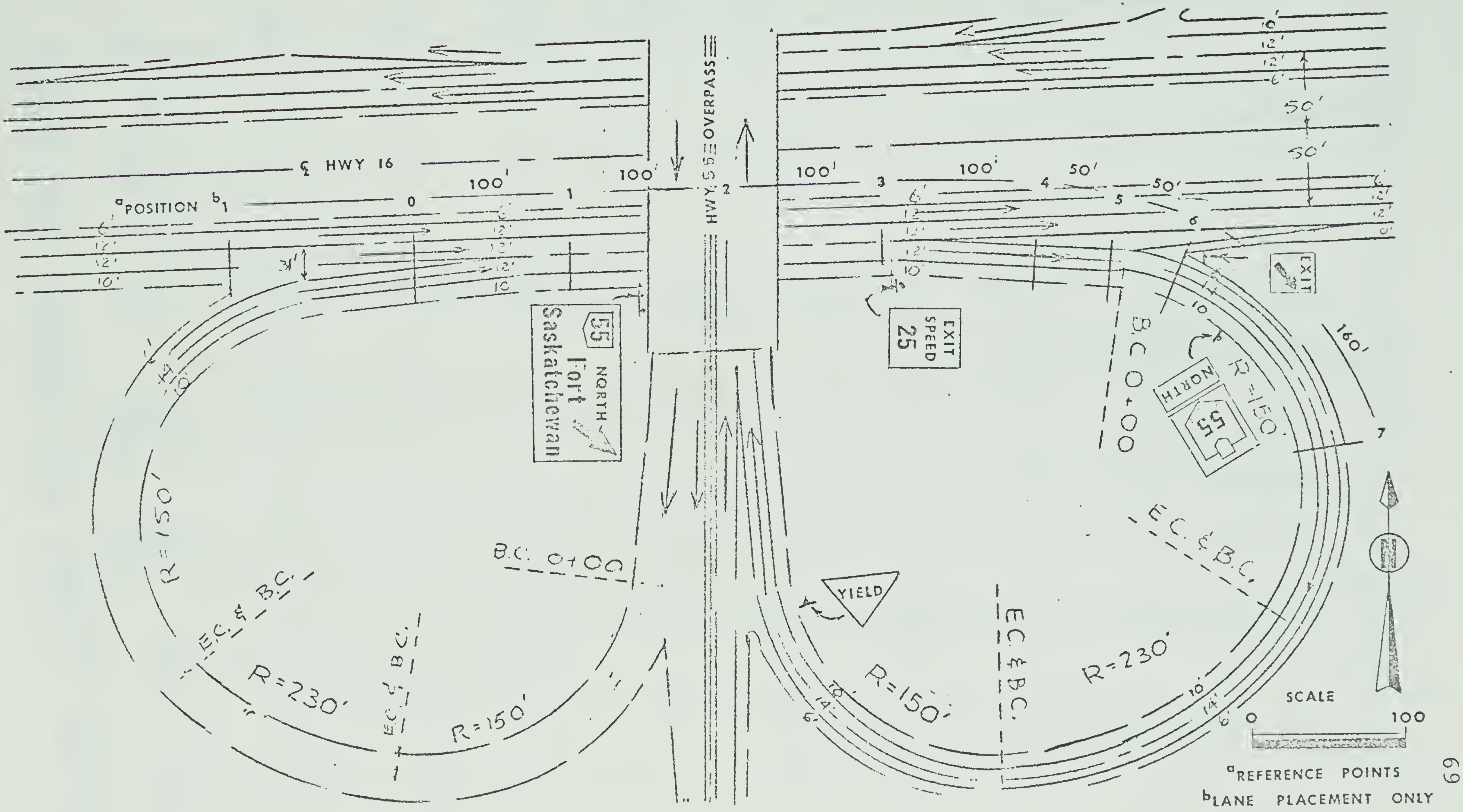

FIGURE VII.5 BREMNER STUDY AREA

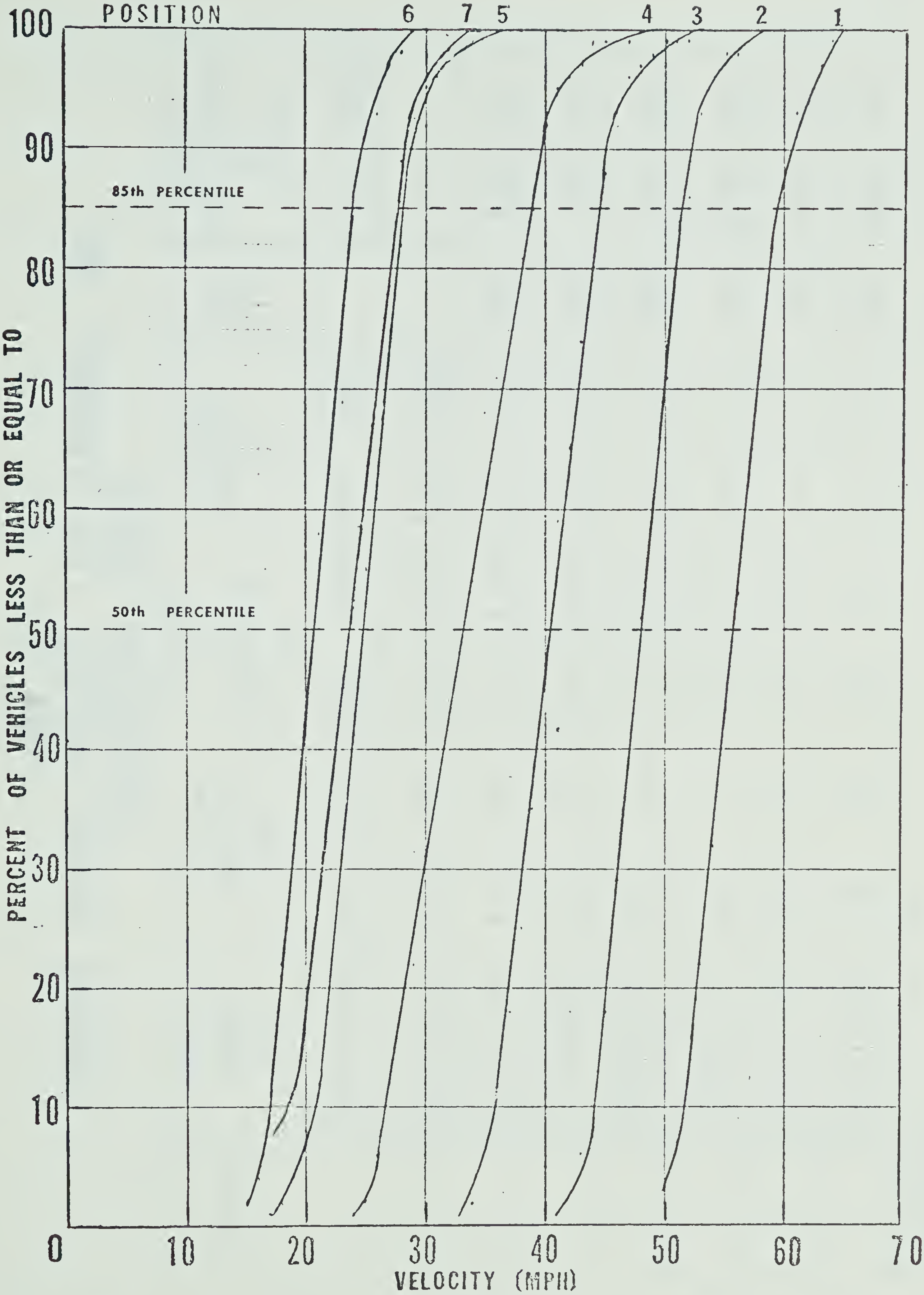

FIGURE VII.6 BREMNER CAR VELOCITY DISTRIBUTION

TABLE VII.3
BREMNER AVERAGE VELOCITY, LANE PLACEMENT AND DECELERATION VALUES

POSITION	AVERAGE VELOCITY (MPH)		STANDARD DEVIATION (MPH)		AVERAGE LANE PLACEMENT (FT)		STANDARD DEVIATION (FT)		DIST. (FT)	AVERAGE DECELERATION RATE (FPSPS)	
	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS		CARS	TRUCKS
1	56.44	49.17	3.42	3.85	11.21	10.42	2.41	2.88	–	–	–
2	48.64	43.86	3.33	3.84	14.71	13.83	2.41	2.54	100	8.82	5.31
3	41.13	38.47	3.77	3.87	13.44	12.78	2.22	2.44	100	7.25	4.77
4	33.79	33.22	5.11	4.30	12.39	12.00	2.22	2.31	100	5.91	4.05
5	25.42	27.35	3.47	3.07	9.60	9.10	2.11	2.26	50	10.66	7.65
6	21.32	21.80	3.21	3.25	6.92	6.27	1.89	2.10	50	4.12	5.87
7	24.19	20.07	4.03	3.56	4.99	4.25	1.47	0.96	160	-0.88	0.49

[a] TWO DECIMAL PLACES ARE NOT MEANT TO IMPLY DEGREE OF ACCURACY; ARE FOR PURPOSES OF STATISTICAL ANALYSIS ONLY

21 mph at the ramp entrance (i.e. position six) to 24 mph at position seven, 160 feet further around the first ramp curve.

The standard deviation of the average car velocities was approximately 4 mph indicating most drivers travelled at a similar velocity on the deceleration lane and the ramp.

The average car deceleration rates are also shown in TABLE VII.3. Generally, the deceleration rates were high. The greatest deceleration (i.e. 10.66 ft/sec^2) occurred in the 100 feet prior to the ramp as vehicles pass the embankment and drivers obtain a view of the ramp curvature.

Truck speeds across the seven reference points are shown in FIGURE VII.7. The 85 percentile truck speed decreased from 53 mph at the start of the deceleration lane (i.e. position one) to 31 mph near the end of the deceleration lane at position five. The 85 percentile speed of trucks on the ramp was 24 mph. The lack of trucks travelling at the high or low extremes resulted in speed curves which are almost straight.

The average truck velocity rates are presented in TABLE VII.3. They were lower than those of cars and decreased from 49 mph at the start of the deceleration lane to 27 mph near the end of the deceleration lane. The average truck velocity at the ramp entrance (i.e. position six) was 22 mph and decreased to 20 mph, 160 feet further around the ramp curve. The increase in grade apparently caused the

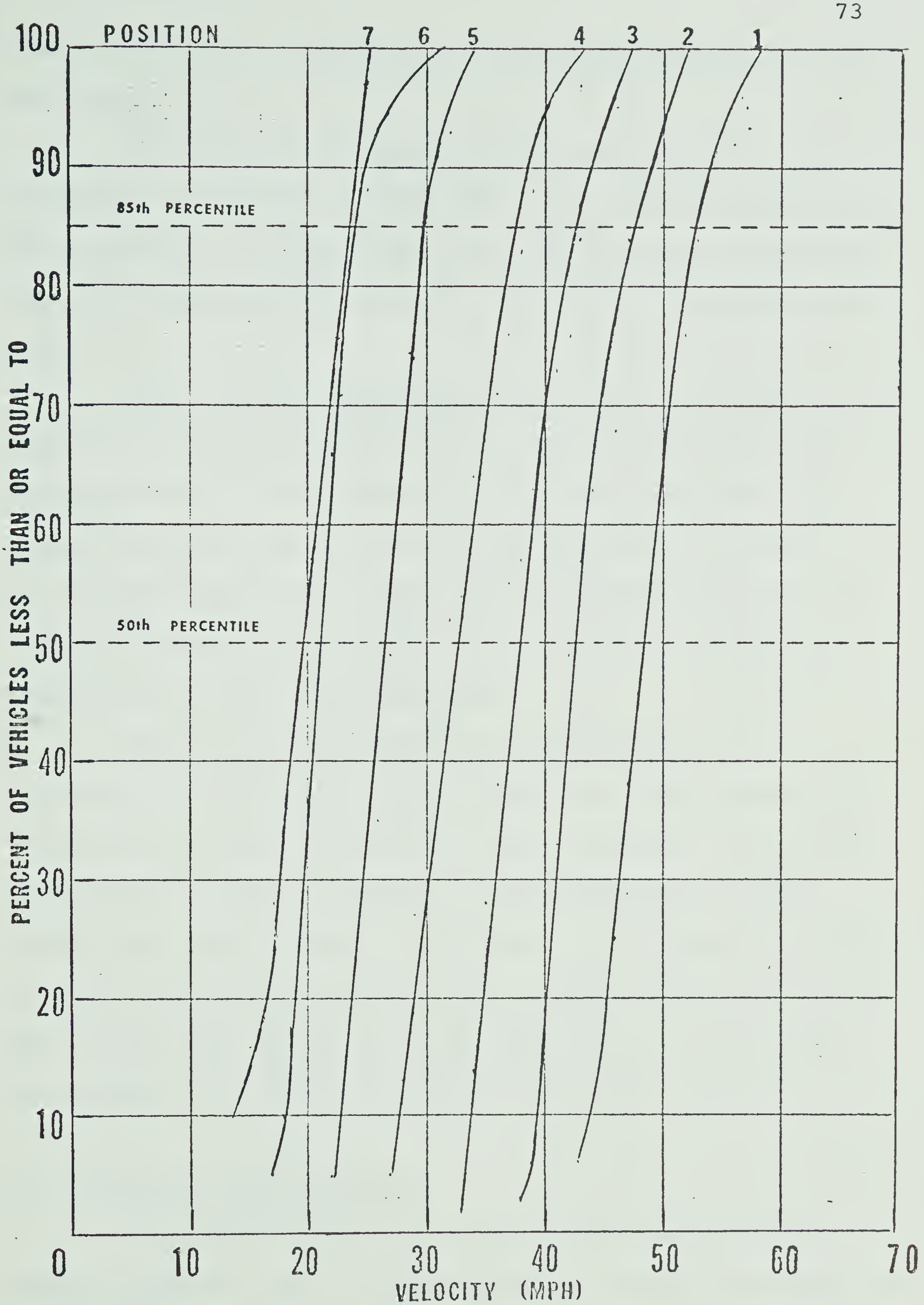

FIGURE VII.7 BREMNER TRUCK VELOCITY DISTRIBUTION

truck velocities to decrease as the trucks proceeded around the curve.

The standard deviation of the average truck velocities, as shown in TABLE VII.3, was approximately 4 mph. This appears to indicate that most truck drivers travelled near the average velocity on the deceleration lane and the ramp.

The average truck deceleration rates, as shown in TABLE VII.3, were smaller and more uniform than those of passenger cars. The greatest deceleration rate was 7.65 ft/sec^2 with an average rate of approximately 5 ft/sec^2. It should be noted that these values approach the limit of 9 ft/sec^2 stated by the AASHO (1965) to be a comfortable maximum for drivers and passengers.

The velocities of both cars and trucks are indicative of driver familiarity with the ramp because all were near the posted limit of 25 mph. However, the posted exit speed is low and appears to be suggestive of urban rather than rural design. Furthermore, the speed differential of 40 mph which exists between the highway speed limit of 65 mph and the exit speed of 25 mph appears to be too great and accounts for the high deceleration rates observed.

7.3.3 Bremner Lane Placement

The lateral position of the right wheels of the Bremner cars is shown in FIGURE VII.8. Position one for this

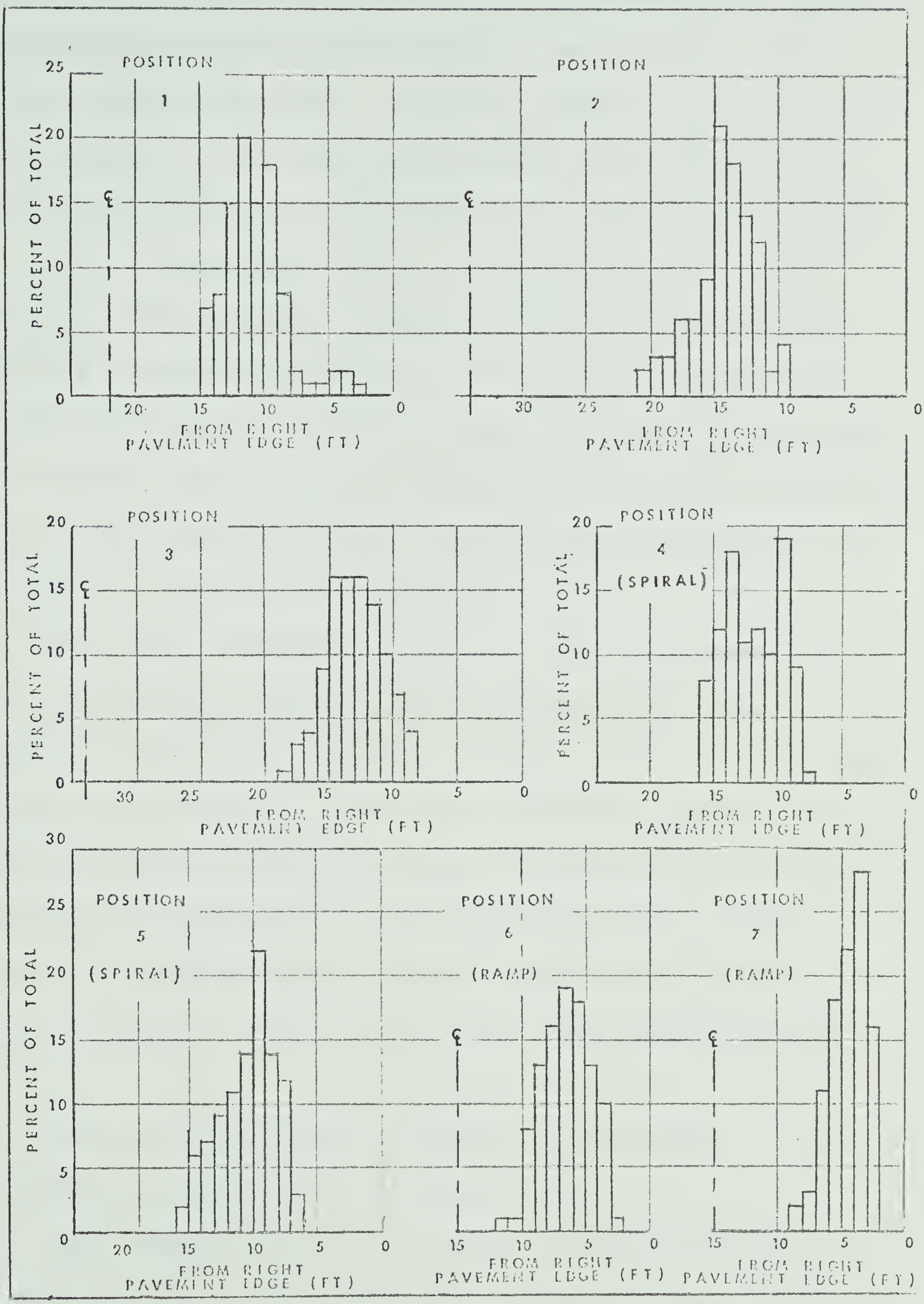

FIGURE VII.8 BREMNER CAR LANE PLACEMENT

[a]10 FOOT RIGHT SHOULDER EXISTS AT ALL POSITIONS

analysis is at the merge point where the pavement of Highway 16 meets the pavement of the southwest inner loop. Other positions remain unchanged. A 10 foot paved right shoulder exists throughout the study area and is indicated by a yellow pavement marking 10 feet from the right pavement edge.

At position one, 35% of the passenger car traffic travelled on the shoulder. This occurred when a diverging vehicle was closely followed by a through vehicle in lane one. The 85 percentile value of lane placement for passenger cars decreased from 17 feet at position two on the deceleration lane to 14 feet from the right pavement edge at position five near the end of the deceleration lane.

The right shoulder of the ramp was travelled by 90-100% of the ramp traffic. The 85 percentile lane placement of the right rear wheel at the entrance to the ramp (i.e. position six) was 9 feet from the right pavement edge. At 160 feet further along the ramp, the 85 percentile value for cars was 6 feet from the right pavement edge. This indicates a disregard of the yellow shoulder marking and the desire of drivers to take the shortest path around a single curve.

The arithmetic mean and standard deviation of the lane placement is provided in TABLE VII.3.

The average car lane placement at position one was approximately 11 feet from the right pavement edge. The remaining average lane placements decreased from 15 feet at position two near the beginning of the deceleration lane

to 10 feet at position five near the end of the deceleration lane. These values relate the position of the right front wheel to the right pavement edge. The average car lane placement at the ramp entrance (i.e. position six) was 7 feet from the right pavement edge and 5 feet at position seven, 160 feet further around the ramp curve.

The standard deviation of car lane placement, from TABLE VII.3, was approximately 2 feet, indicating a similar path was followed by all drivers travelling on the deceleration lane and the ramp.

The truck data for lateral position on the ramp is inconclusive with only four trucks available for analysis. Generally, as shown in TABLE VII.3, trucks travelled closer to the right pavement edge than did the passenger cars. At position one, average truck lane placement was 10 feet from the right pavement edge. Average truck lane placement decreased from 14 feet at position two near the beginning of the deceleration lane to 9 feet at position five near the end of the deceleration lane. At the ramp entrance (i.e. position six), the average truck lane placement was 6 feet and decreased to 4 feet at position seven, 160 feet further around the first ramp curve.

From TABLE VII.3, the standard deviation of the average truck lane placement was approximately 2 feet, indicating a similar path was followed by all truck drivers as they travelled on the deceleration lane and the ramp.

Generally, the auxiliary lane was not well used with vehicles decelerating partially on the through lanes. From the results, it can be seen that drivers tended to aim for the beginning of the ramp curve and proceeded around the ramp as close as possible to the right pavement edge.

7.3.4 Bremner Statistical Analysis of Velocities

Only the car and truck velocities of positions 4-7 were compared to determine if there was any significant difference between them. These were chosen because they were located on the spiral curve (i.e. positions 4 and 5) and the study ramp (i.e. positions 6 and 7), which were the most important areas of the study.

The results of the "F test" and the "t test", as shown in APPENDIX D, indicate there is a significant difference between car and truck velocities at positions 5 and 7 at the 5 percent level of significance. Apparently, at position five, 50 feet in advance of the ramp entrance, trucks have a greater momentum than cars and, consequently, travel at a higher velocity. Trucks continue to decelerate at a higher rate than cars until, at position seven, the truck velocity is lower than that of cars because of the increase in grade (i.e. from 2 to 4 percent between positions five and seven). Obviously, the greater ease of mobility of cars accounts, in large part, for the significant difference between the car and truck velocities.

7.3.5 Bremner Weaving Maneuvers

Because of few vehicles on the inner loop connecting Highway 55 from the north to Highway 16 to the east, there were few weaving maneuvers. The weaving maneuvers that did occur involved no hazardous movements.

The vehicles entering Highway 16 did not, however, make full use of the auxiliary lane. They travelled in a straight path on a tangent to the final curve on the south-west inner loop.

The vehicles leaving the through lanes of Highway 16 did not make full use of the auxiliary lane either.

7.3.6 Bremner Geometrics

The geometrics of the Bremner study ramp as compared to recommended design practice are presented in APPENDIX E, TABLES E.2A, E.2B and E.2C.

The most important geometric feature of the ramp is a ramp design speed which is too low for a highway with through lanes constructed to freeway design standards. In addition, the auxiliary lane length is insufficient to permit a comfortable deceleration from the posted highway speed limit to the exit ramp speed. The sight distance is also restrictive since the overpass embankment does not permit a view of the ramp until the embankment has been passed. The large degree of curve intensifies this problem as drivers do not realize how small the loop is until they have passed the

embankment and are entering the ramp.

The results obtained with the ball bank indicator from trial runs on the Bremner study ramp are summarized in TABLE VII.4.

The substantial increase in the ball bank angle reading over a 5 mph increase in exit speed indicates a high centrifugal force and body roll angle. Furthermore, with a curve ratio, that is the ratio of the larger ramp curve to the smaller ramp curve, of 1.53 (i.e. 230'/150'), a considerable amount of steering effort is required to remain on the ramp pavement.

Further evidence of a high centrifugal force on the first ramp curve is the black appearance of the left 15 feet of the ramp pavement, the result, no doubt, of tire rubber wear while drivers negotiate the initial ramp curve. It is especially black in the middle of the pavement, indicating a high centrifugal force and body roll angle cause large forces to be applied to the left wheels of the vehicles travelling the first ramp curve. The remainder of the ramp is not as black as the first portion.

Three other observations were noted by the author during the use of the ball bank indicator.

First, pavement markings in the deceleration and right shoulder ramp area are completely ignored by the experienced drivers and have been obliterated.

Second, because the ramp pavement has a smooth

TABLE VII.4

BREMNER BALL BANK INDICATOR VALUES

SPEED MPH	BALL BANK RDG., DEGREES		CORRESPONDING [b] "f" VALUES	
	[a]RECMMD.	TEST	[a]RECMMD.	TEST
25	12	14 14	0.18	0.16
30	12	20 20	0.15	-

a RECOMMENDATIONS FROM AASHO (1965), p. 154

b TRANSVERSE COEFFICIENT OF FRICTION

texture caused by asphalt bleeding and the removal of tire rubber, it is difficult to distinguish the intended ramp lane from the right shoulder of the ramp.

Third, the "EXIT SPEED 25" sign is placed at the beginning of the spiral, only 150 feet in advance of the ramp. This placement coupled with a restricted sight distance because of the overpass embankment, would ordinarily result in high exit speeds. However, because drivers familiar with the ramp were observed, no difficulty was encountered during the film analysis.

7.3.7 Bremner Accident Record

Between 1968 and 1970, there were 3 single-vehicle accidents on the Bremner study ramp. In each case, a vehicle skidded on an icy or oily surface and left the roadway.

Apparently, a low coefficient of friction and a large degree of curvature coupled with an exit speed probably in excess of the posted limit of 25 mph caused the accidents.

7.4 Groat Road and 107 Avenue

7.4.1 Groad Road Volume

There were 730 merging vehicles observed during one hour. Of these, 35 or 5% were trucks. Prior to the merge, there were 980 through vehicles on Groat Road, 22

or 2% of these were trucks. The lane adjacent to the median carried 550 vehicles before the merge, 4 of which were trucks. The distribution of traffic on Groat Road prior to the merge was 44% in the merge lane, 4% composed of trucks; and 56% in the inside lane, 1% trucks.

The distribution after the merge was 1087 vehicles or 64% in the outer lane, 55 or 5% trucks; the lane adjacent to the median carried 623 or 36% of the traffic, 2 or 0.3% were trucks. The truck traffic after the merge accounted for 3% of the total traffic.

7.4.2 Groat Road Velocity

The vehicle speeds were calculated using four reference points as shown in FIGURE VII.9 and APPENDIX A, FIGURE A.3. Position one was near the beginning of the acceleration ramp. Position two was at the nose terminal. Position three was located near the middle of the acceleration lane. Position four was located near the end of the acceleration lane.

The car velocity distribution is shown in FIGURE VII.10. The lower velocities on the acceleration lane were the result of drivers slowing down because they were uncertain whether or not they could accept the gap. In some cases, drivers came to a complete stop on the acceleration lane and waited for a large gap before entering Groat Road. This type of maneuver resulted in queues forming behind the

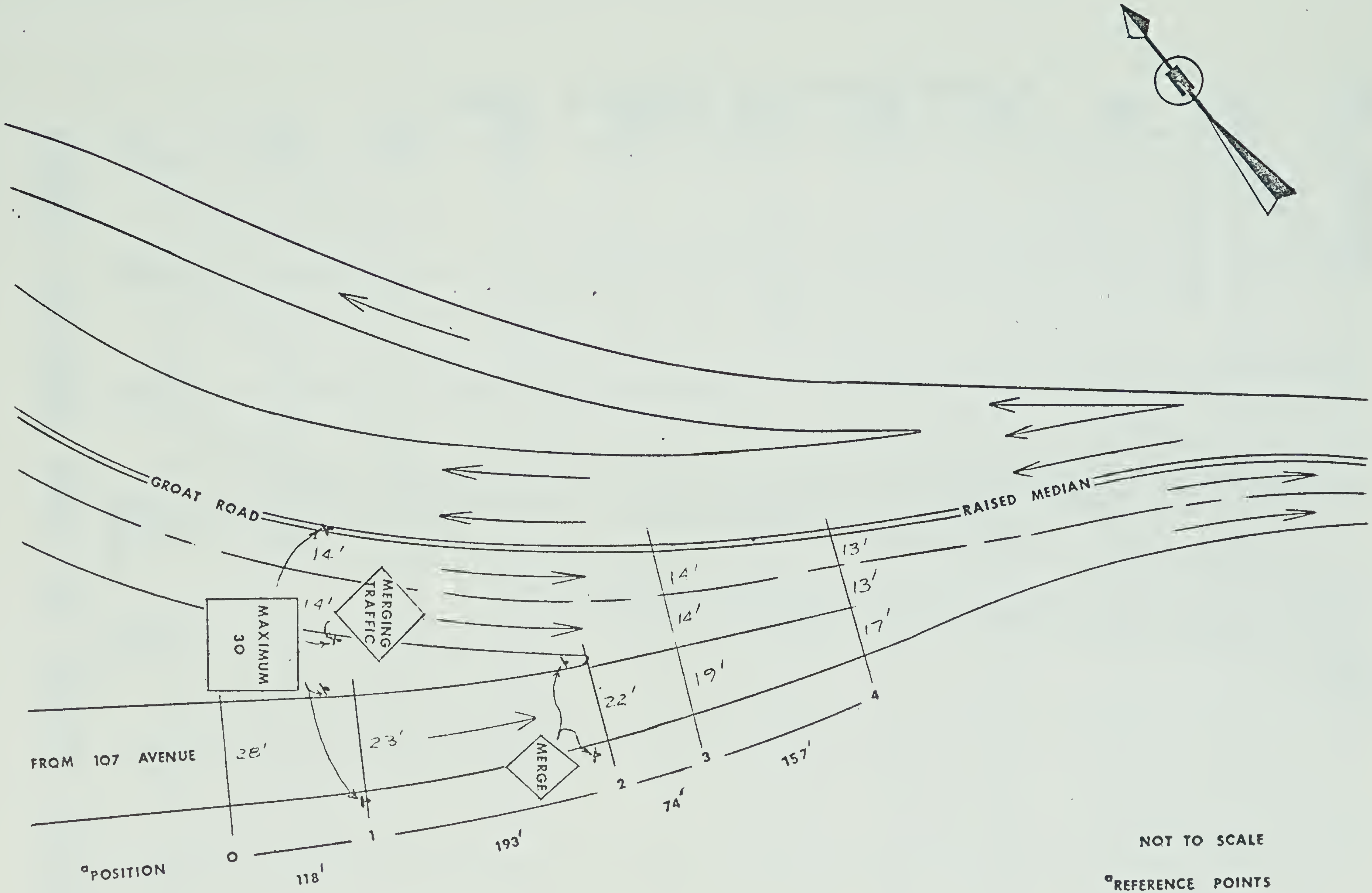

FIGURE VII.9 GROAT ROAD STUDY AREA

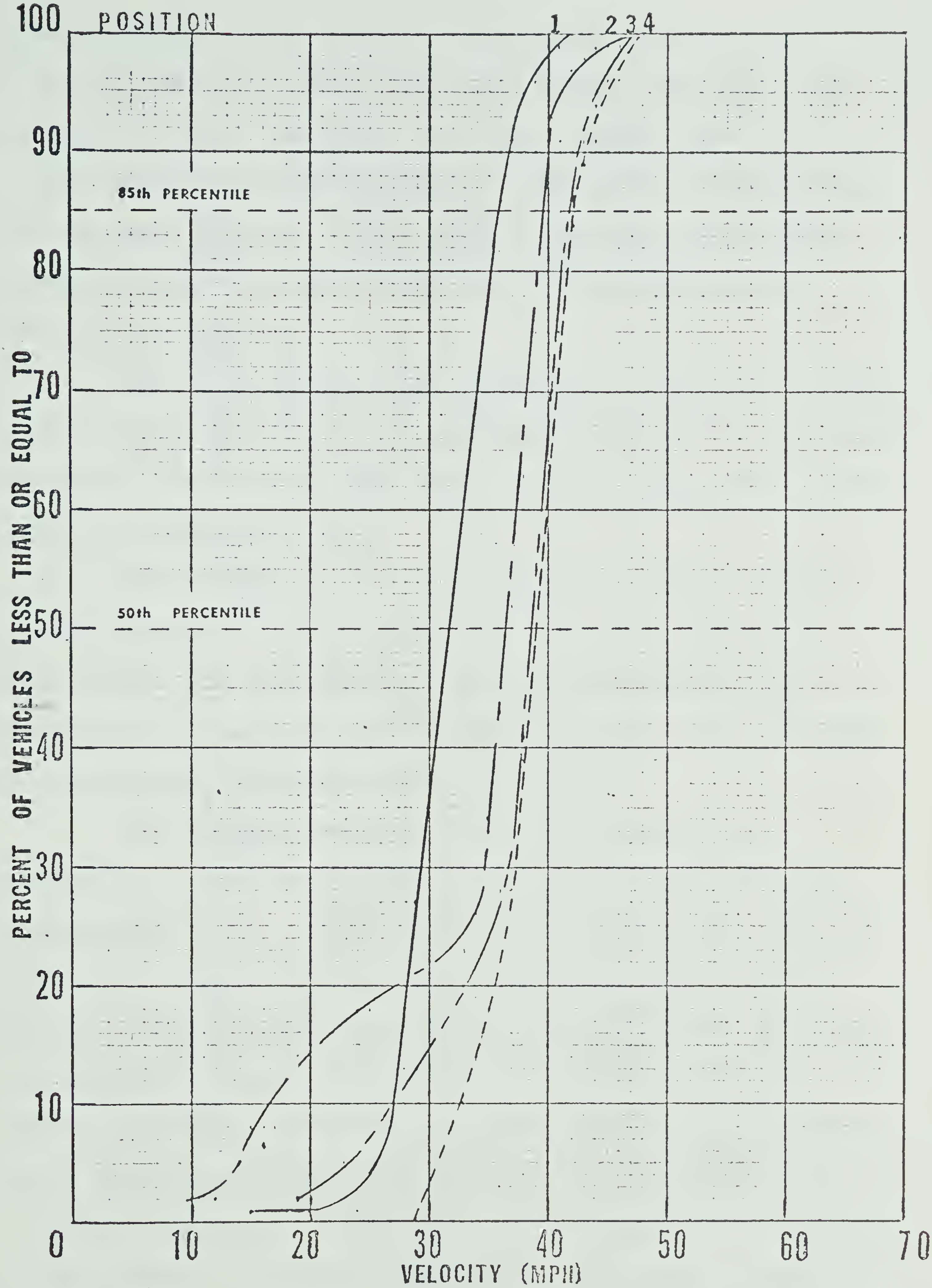

FIGURE VII.10 GROAT ROAD CAR VELOCITY DISTRIBUTION

stopped vehicle. Occasionally, the driver of the third or fourth vehicle in the queue would become impatient, pull out from the queue and enter the outer through lane, or, in some cases, the lane adjacent to the median, immediately past the nose terminal. This type of maneuver was usually performed at low initial velocities and subsequent high rates of acceleration.

The 85 percentile car velocity increased from 36 mph on the initial portion of the acceleration lane (i.e. position one) to 42 mph on Groat Road near the end of the acceleration lane (i.e. position four).

The arithmetic mean and standard deviation of the velocities across the four reference lines are shown in TABLE VII.5. The average car velocities increased from 32 mph at position one near the beginning of the ramp to 39 mph at position four near the end of the ramp.

The standard deviation from the average car velocities at position two (i.e. at the nose terminal) was approximately 9 mph. This large deviation was the result of some drivers decelerating before they merge; while others would accelerate before the merge. These different attitudes toward merging tended to pull values away from the mean. A similar condition occurred at position three (i.e. near the middle of the acceleration lane) with a standard deviation of 6 mph. Positions one and four at the beginning and end of the acceleration ramp, respectively, produced standard

TABLE VII.5

GROAT ROAD AVERAGE VELOCITY, LANE PLACEMENT AND ACCELERATION VALUES

POSITION	AVERAGE VELOCITY (MPH)		STANDARD DEVIATION (MPH)		AVERAGE LANE PLACEMENT (FT)		STANDARD DEVIATION (FT)		DIST. (FT)	AVERAGE ACCELERATION RATE (FPSPS)	
	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS	CARS	TRUCKS		CARS	TRUCKS
1	32.42	29.54	3.99	2.27	11.09	8.37	1.31	1.11	—	—	—
2	34.16	33.06	8.60	2.71	11.58	8.80	1.36	1.08	193	0.65	1.23
3	37.53	35.09	5.92	3.30	20.27	13.49	9.40	2.73	74	3.51	2.01
4	38.90	37.54	3.94	3.16	22.11	14.94	6.88	2.82	157	0.72	1.22

[a] TWO DECIMAL PLACES ARE NOT MEANT TO IMPLY DEGREE OF ACCURACY; ARE FOR PURPOSES OF STATISTICAL ANALYSIS ONLY

deviations of approximately 4 mph. This value would indicate drivers were not travelling at the high or low extremes but were closer to the mean value.

The average car acceleration rates, as shown in TABLE VII.5., were greatest in the area where the actual merge maneuver was made. These rates show a slight acceleration on the initial portion of the acceleration ramp, then a greater acceleration as drivers pass the nose terminal and attempt to remain ahead of the drivers on the through lanes and, finally, a lesser acceleration as drivers from the ramp, having accepted a gap, attempt to maintain their position ahead of those already on the through lanes.

With the greatest average car acceleration rate being 3.51 ft/sec^2, the acceleration rates for cars were well within the comfortable maximum of 9 ft/sec^2 as stated by the AASHO (1965).

The truck velocity distribution curves of FIGURE VII.11 indicate a more uniform speed distribution since few of the observed truck drivers travelled at the high or low extremes. This behavior resulted in curves which are almost straight. The 85 percentile truck speeds varied from 32 mph on the initial portion of the acceleration lane at position one to 41 mph near the end of the acceleration lane at position four.

From TABLE VII.3, average truck velocities varied in an increasing manner from 30 mph at position one to 38 mph at

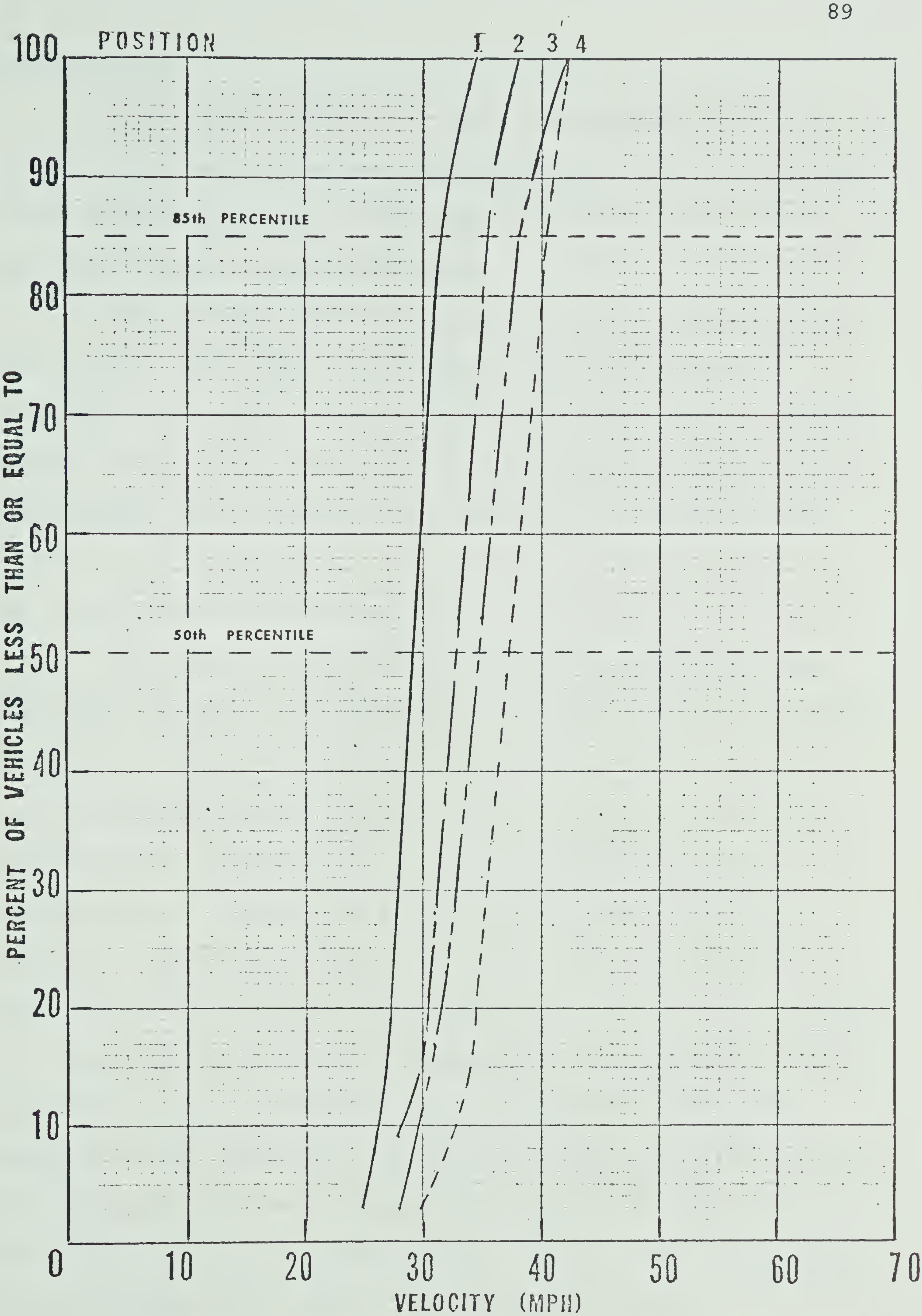

FIGURE VII.11 GROAT ROAD TRUCK VELOCITY DISTRIBUTION

position four.

Also from TABLE VII.3, the standard deviation of average truck velocities was approximately 3 mph, indicating a more uniform driving pattern among truck drivers, with most speeds near the average value.

The average truck acceleration rates, as shown in TABLE VII.5, were also more uniform than those of the passenger cars. The average truck acceleration rates varied between 1-2 ft/sec^2. As would be expected, they were also much smaller than the greatest acceleration rates of cars.

7.4.3 Groat Road Lane Placement

The lateral position of cars with respect to the right curb is shown in FIGURE VII.12. The various histograms show the location of the right front wheel. The results indicate drivers tend to drive closer to the left curb on the acceleration lane (i.e. at positions one and two) in anticipation of entering the through lanes as quickly as possible. Twenty-six percent of the drivers entered the outer lane of Groat Road within 200 feet of the nose terminal, (i.e. position 3) although 15% entered Groat Road by swinging over into the lane adjacent to the median once they had passed the nose terminal. At position four, near the end of the acceleration lane, 157 feet from position three, 70% of the traffic had entered Groat Road, leaving only 30% of the traffic to make full use of the acceleration lane.

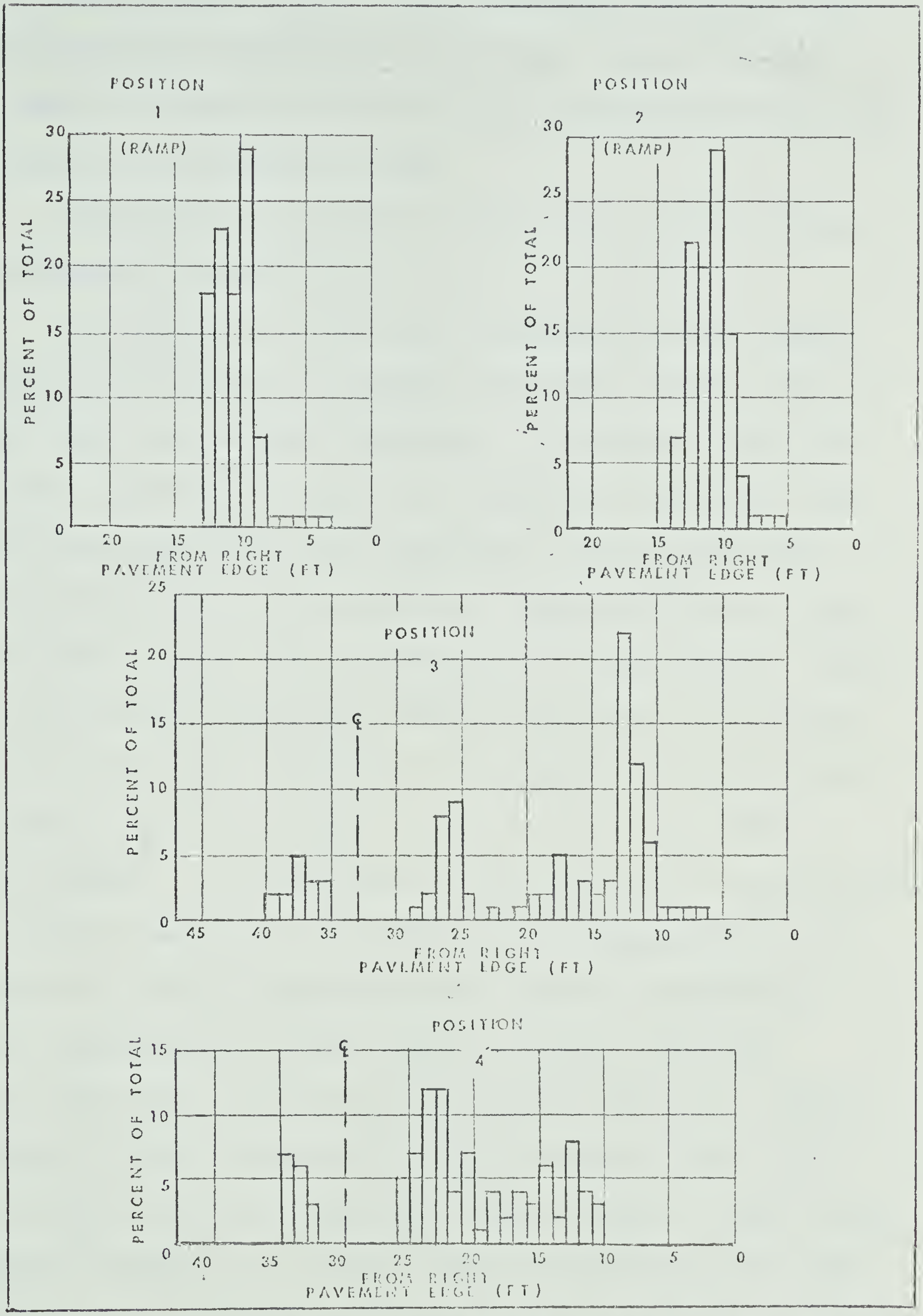

FIGURE VII.12 GROAT ROAD CAR LANE PLACEMENT

[a]NO SHOULDER PROVIDED AT ANY OF THE POSITIONS SHOWN

The merging truck traffic entered the outer lane of Groat Road and remained in that lane. Truck drivers also tended to keep to the left of the acceleration lane in anticipation of accepting a gap.

Average lane placement values of cars and trucks are included in TABLE VII.5.

It should be noted that, although average values are usually of concern in studies involving numbers, the average value of the lane placement at the Groat Road and 107 Avenue southbound merge is an ambiguous statistic once drivers have passed the nose terminal. Merging drivers either travelled on the acceleration lane, pulled into the outside lane or into the lane adjacent to the median. With such a variation in possible vehicle locations, an average value is not of much relevance. The standard deviation of car lane placement, as shown in TABLE VII.5, provides a perfect example. At positions one and two on the acceleration lane prior to and at the nose terminal, respectively, the standard deviation is approximately one foot indicating drivers travelled in similar paths at these locations. However, once the car drivers have passed the nose terminal, the deviation from the mean is approximately 9 feet and 7 feet at positions three and four, respectively. These large deviations indicate car drivers did not follow in the same or similar paths while merging and merged according to their own judgement. The standard deviation of average truck lane

placements were more uniform and indicate truck drivers do not have the maneuverability of cars in a merge situation. The standard deviation of average truck lane placement varied between one and three feet.

7.4.4 Groat Road Statistical Analysis of Velocities

The velocities of cars and trucks as they crossed all four reference positions were analyzed. The results of the "F test", as shown in APPENDIX D, indicate a rejection of the null hypothesis. Car and truck velocities are, therefore, significantly different at the merge on Groat Road at 107 Avenue. This would be due to the different acceleration characteristics between passenger cars and trucks because,with a greater weight-horsepower ratio, trucks require a longer distance to accelerate to any given velocity than cars.

This concludes the presentation of the results of the research. The results were obtained using methods that could, in themselves, cause errors. Overlays become increasingly more suspect as the distance from the camera increases. The width of a pencil line, for example, could represent 5 feet at a distance of 1000 feet from the camera. However, the author took great care to insure that error of this nature was kept to a minimum by drawing overlay lines with fine pointed pens. The particular method used to

analyze the film (i.e. employing the analyzer projector and screen inlaid into a desk top) also enabled the author to obtain results as good as could possibly be obtained from such a photographic study. A seated position is the most natural and reduces the strain and boredom of standing at a vertical screen. Fatigue can lead to errors.

Furthermore, because the results of this study correlate with those of other similar studies and because the correlation was not checked until after the results of this study were obtained (to reduce bias), the author believes the results of this study contain errors within the usual range of human capability and do not differ significantly from the actual condition.

CHAPTER VIII

CONCLUSIONS

The purpose of this research was to investigate the traffic operating characteristics at two interchanges near Edmonton. In addition, the geometric design of the interchanges was also to be considered. It was anticipated that the study would provide an explanation of the causes of single-vehicle accidents at these and similar interchanges in Alberta. A third interchange was later studied to obtain information on the merging ability of Alberta drivers.

The research was carried out by employing a photographic technique and analyzing volume, vehicle speed, lane placement and weaving maneuvers of traffic using preselected study ramps.

The major results include a tendency for drivers to aim for the beginning of an exit ramp curve, decelerating partially on the through lane; a disregard of posted exit speed signs with a large percentage of drivers travelling in excess of posted speed limits, and, finally, a tendency for drivers to travel on the right shoulder of the exit ramp.

This chapter presents the conclusions based on the results of the research.

8.1 Photographic Technique

The photographic method of traffic analysis is a good one. Vehicle speeds, lane placement, volume and weaving maneuvers are readily available from film of traffic movements. The photographic equipment does not influence the majority of drivers. This conclusion is based on the experience of the author and a review of the literature.

Similarly, it can also be concluded that white reference lines painted on the shoulder of the road do not influence drivers. However, white lines for reference points for speed or lane placement are of no value unless they are close to the camera (i.e. within 600 feet) and unless they are sufficiently large to be readily visible on the film (i.e. greater than 6 inches wide and 3 feet long).

When numerous study sites are analyzed, a mobile vehicle, such as a "giraffe", provides an excellent means of obtaining an aerial view of the study areas. Depending on the topography and area to be filmed, a height of 30-40 feet above the ground provides an adequate aerial view.

Results are readily attainable up to a distance of 600 feet and, therefore, only vehicles within 600 feet of the camera should be analyzed. Thereafter, the size of the vehicle in proportion to the frame size decreases as the distance from the camera increases and it becomes increasingly more laborious to analyze vehicle movements. Newman (1963) reached a similar conclusion and stated " . . . detailed

measurements can be made for sections up to 400 feet, whereas 700 to 1000 feet of roadway can be viewed subjectively."

8.2 Salisbury

Based on observed volumes of approximately 1200 vehicles per hour on the highway and a design capacity of 2000 vehicles per lane per hour, there is adequate capacity at the Salisbury interchange for many years to come.

Because of the amount of shoulder driving on the ramp, it can be concluded that drivers tend to reduce the curvature by taking the shortest path around the curve and, so as to straighten the curves out, aim for the inside of the second curve. Similar results and conclusions were recorded by Davis and Williams (1968). The faster drivers cut the corner more than the slower ones.

Based on the analysis of the Salisbury study ramp geometrics, the radius of curvature of the ramp is too small to be safely travelled at speeds in excess of 35 mph. Since 40% of the traffic at the ramp entrance and 70% of the traffic at the end of the first ramp curve travel faster than 35 mph, it can be concluded that this is not a safe condition.

It can also be concluded that the yellow marking on the right shoulder of the ramp does not influence drivers. They generally tend to drive at whatever position on the pavement gives the least resistance to the speed they are travelling (i.e. usually close to the right pavement edge).

It can be concluded that the absence of a spiral curve into the ramp causes a large centrifugal force at the ramp entrance. This is shown by the increase in the angle reading of the ball bank indicator as the exit speed increases (see TABLE VII.2). Drivers must veer to their right in order to remain on the pavement. This fact, coupled with an exit speed probably in excess of the posted limit of 30 mph, leads one to the conclusion that this is the reason drivers fail to negotiate the first curve on the ramp.

With high ramp speeds one must conclude that drivers are not taking full advantage of the deceleration lane. In addition, with a highway design speed between 60-70 mph, a ramp design speed between 30-50 mph is recommended. This condition is satisfied with the design of the study ramp (i.e. 30 mph). However, it must be concluded that the Salisbury interchange ramp studied is designed on the basis of minimum acceptable limits (i.e. according to AASHO (1965)) which do not necessarily encompass the safest design practice.

8.3 Bremner

With a total highway volume of 856 vehicles over a four hour filming period and a design capacity of 2000 vehicles per lane per hour, there is adequate capacity at the Bremner interchange for many years to come.

Based on ramp speeds obtained, it can be concluded that drivers observed are familiar with the ramp since

all speeds are near the posted exit speed of 25 mph. The importance of driver experience in interchange operation is corroborated by the HIghway Capacity Manual (1965) which states " . . . interchanges carrying predominately commuter traffic tend to have smoother operating characteristics than those carrying the same volume of tourist or long-distance traffic." The reduction in the number of single-vehicle accidents several years after the opening of the Salisbury interchange also indicates a better type of operation as drivers become more experienced with the interchange.

An exit speed of 25 mph is, however, too low for a highway with a posted speed limit of 65 mph. The speed differential of 40 mph is too great and it can be concluded that according to AASHO (1965), an exit speed of 25 mph is indicative of urban rather than rural design.

Based on the results of the lane placement analysis, the ball bank indicator readings and the black appearance of the first ramp curve, it must be concluded that the first ramp curve, with a radius of 150 feet, is too small because almost 100% of the drivers travel on the right shoulder of the ramp at low speeds (i.e. 20-25 mph).

Based on the analysis of the Bremner study ramp geometrics, it can be concluded that there is an insufficient deceleration lane length causing high rates of deceleration (i.e. as high as 11 ft/sec^2). The auxiliary lane was not used to full advantage and drivers decelerated partly on the outer

through lane and aimed for the beginning of the ramp curve. Davis and Williams (1968) also concluded that drivers aim for the beginning of a ramp curve.

In addition, it can be concluded that high deceleration rates which occur prior to the exit intensify the need for a high coefficient of friction to prevent vehicles from skidding while decelerating.

Based on the rough estimates of the side friction obtained with the ball bank indicator, it can be concluded that the coefficient of friction (see TABLE VII.4) is inadequate for speeds greater than the posted exit speed. The importance of the small coefficient of friction and low exit speed is exemplified by the accidents which occurred on the study ramp. It must be concluded that icy or oily conditions and an exit speed probably in excess of 25 mph resulted in skids and single-vehicle accidents.

Since the left 15 feet of ramp pavement was rarely used, it can be concluded drivers tend to take the shortest path possible around ramp curves. Davis and Williams (1968) also reached a similar conclusion.

8.4 Groat Road and 107 Avenue

Because full use was not made of the acceleration lane, it must be concluded that drivers keep to the left of the acceleration lane so that they can enter the through lanes as soon as possible (i.e. usually within 200 feet of the nose

terminal). Driving with reference to the left-hand side was also found by Keese, Pinnell and McCasland (1960). Newman (1963) also discovered drivers attempting to enter the through lanes as soon as possible once they have passed the nose terminal. He concluded re-striping improved the use of the acceleration lane.

It must be concluded that the willingness of drivers to either pull into the inside lane on Groat Road prior to the merge or to slow down and permit a sufficiently large gap for merging traffic resulted in a smooth merge operation. The overall merge maneuver was performed extremely well with less than one percent of the drivers coming to a complete stop on the acceleration lane. It must be concluded, therefore, that a merge only performs well when drivers on the through and merge lanes drive courteously and are experienced with a merge maneuver.

This completes the presentation of the conclusions. The conclusions of this study are based on and limited to observations of traffic during dry, summer, morning peak hours. This study would, therefore, encompass drivers commonly classified as commuters and the conclusions reached are limited primarily to drivers familiar with the interchanges studied. Furthermore, the conclusions reached apply only to those ramps that were specifically studied. However, ramps of similar design should have similar operating

characteristics and should produce similar results and conclusions.

The conclusions of a study are only as good as the results obtained. Since the results correspond to those of other similar studies, it can be concluded that the results and conclusions of this study are of some merit.

CHAPTER IX

RECOMMENDATIONS

This chapter presents the recommendations of the author based on the results and conclusions of the research.

9.1 Photographic Technique

It is recommended that, in any future photographic studies, a detailed analysis of traffic movements be restricted to a distance of 600 feet. The author did not attempt to use a telephoto lens and the analysis difficulties encountered beyond a distance of 600 feet might be overcome by its use. Distortion could be a problem with such a lens.

The use of objects adjacent to the study area provide an excellent means of referencing for distance calculations. By having reference points on the film, the position of the projector is not relevant because distances between poles, for example, are always constant.

A "giraffe" with a generator would be a useful piece of equipment; but, if no generator is provided, an inverter and battery system works well. It is recommended that, for any repetitive filming session (e.g. one hour per day for five days), a fully charged battery be used at the start of each days filming.

It is also recommended that future photographic

studies be undertaken to analyze traffic movements during the off peak hours when commuters do not form a large percentage of the traffic.

9.2 Study Interchanges

Since Alberta highway through lanes are generally designed to expressway and freeway standards, it is recommended that interchanges which permit an exit speed greater than 25-30 mph be constructed. Inner loops with a greater radius of curvature and introductory spiral curves would provide the means of attaining higher exit speeds. In a study performed by Gray and Kauk (1968), which compared circular and elongated loop ramp alignment, it was concluded that " . . . a circular loop ramp alignment is more conducive to ease of operation than an elongated alignment. Consequently when a loop ramp is being designed and relative economic and operational factors are being assessed, operational preference should be given to a circular alignment."

It is recommended that the reverse curve design of outer connections should be avoided since drivers do not generally travel between the shoulder markings of a ramp of this type. In addition, since the left half of the ramp pavement is seldom used a lesser pavement width would be adequate (e.g. 22 feet). A lesser ramp pavement width could also provide a funneling effect which has been known to cause a reduction in vehicle speeds.

Similarily, since drivers usually travel on the right shoulder of the ramp, it is recommended that the right shoulder should be smaller than the left shoulder. A right shoulder of 10 feet encourages a vehicle to stop. However, most vehicles travel on this portion of the ramp and a vehicle stopped on the right is a hazard. Furthermore, few vehicles stop on the shoulders of a ramp.

In view of the low volumes on Alberta highways and the better operating characteristics of diamond interchanges, it is recommended that diamond interchanges be constructed in areas where volumes are anticipated to remain low for some time to come. Diamonds are the simplest, cheapest and require the least space. Designed with foresight, diamonds can be constructed to accommodate future through lane additions and can have a capacity greater than the cloverleaf. Loutzenheiser (1969) states that today's design of rural diamond interchanges has the crossroad terminals further removed from the structure than did the earlier ones, with distances of 300 to 500 feet being about double previous designs.

It is recommended that pavement markings be maintained on interchanges. Unfamiliar drivers require guidance in choosing the proper lane in interchange areas and any indecision resulting from a lack of adequate signing or pavement markings could result in a hazardous maneuver and possibly an accident. This is especially true of the Bremner

interchange where pavement markings in heavily travelled areas have been obliterated.

In addition, it is recommended that the exit speed sign at Bremner should be placed further in advance of the exit (e.g. 200-250 feet), to provide increased warning of the low exit speed.

Since most drivers, especially at the Salisbury interchange, did not obey the recommended exit speed signs, it is recommended that the sign wording be changed to an advisory ramp speed with the numerical value the safe design speed of the exit ramp and placed so as to permit comfortable deceleration to this safe speed.

Since deceleration lanes have generally not been effectively employed by drivers, a trial use of rumble strips is recommended. Rumble strips in the past have been noted to work effectively by reducing speeds and accidents. In a report by Kermit (1968), a complete description of the rumble strip installed in Contra Costa County (California) is provided. Kermit (1968) states:

"The strips consisted of a sandwich of $\frac{1}{2}$" - $\frac{3}{4}$" aggregate between two layers of polyester resin. The unique feature of the rumble strip developed by Contra Costa County is that it consists of intermittent strips of noisy pavement. The extent of the strip depends on the design speed of the road. The distance between individual strips is variable and decreases as the driver approaches the problem location. If

he heeds the message of the rumble strips, i.e. noise vibration of the vehicle, and slows down, the beat of the strips is constant. If he continues at his former speed, the beat of the strips becomes faster, thereby transmitting a sense of urgency. It is this particular arrangement that, in our opinion, has made our rumble strips so effective."

A rumble strip, such as this, placed along the deceleration lane of ramps with low exit speeds could possibly provide a remedial measure and help reduce the number of single-vehicle accidents.

Because it appears that a low coefficient of side friction, as roughly determined with the ball bank indicator, was one of the major causes of the single-vehicle accidents at the rural interchanges, it is recommended that the coefficient of friction be increased. Surface treatments or seal coats are two possible remedial measures. Perhaps, a newer method, that of safety grooving, could be employed. Safety grooving consists of cutting shallow, longitudinal grooves in the pavement with diamond segmented blades mounted on a rotating shaft. Test results (Concut Inc., 1967) of safety grooved sections have shown a substantial increase in the coefficient of friction and a drastic reduction in the number of skidding accidents.

In addition, it is also recommended that merge signs be placed on rural interchanges wherever possible. This would provide Alberta drivers with an opportunity to learn how to

merge properly while volumes are still low. The outer connections of most interchanges could accommodate a merge.

Eventually, volumes will increase and numerous difficulties will be encountered at interchanges with yield signs. Drivers are reluctant to merge from a ramp having a yield sign and usually come to a complete stop if any traffic is approaching on the through lanes.

These remarks are based solely on observations of vehicle movements at merge points during filming of diverge maneuvers in rural areas. Generally, there were few opportunities for a merge; however, whenever a merge did present itself, the vehicle on the ramp would invariably yield. Experience in a merge situation is a necessary prerequisite for a smooth merge maneuver, as exemplified by the results of the merge considered at Groat Road and 107 Avenue. According to R. H. David, former Assistant Traffic Engineer for the City of Edmonton, drivers initially had a substantial amount of difficulty merging at this location. However, as driver experience increased, the merge operated increasingly better, until it is at its present level of smooth operation.

Therefore, the author believes Alberta drivers should be given a chance to learn how to merge properly at high speeds while highway volumes are still relatively low.

This concludes the recommendations proposed by the author. Spiral curves and larger curve radii, permitting

greater exit speed, on cloverleaf-type interchanges and the construction of more diamond-type interchanges would alleviate the majority of the difficulties encountered by most drivers on Alberta interchange ramps.

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APPENDIX A

TEST LOCATIONS

A.1 Salisbury Interchange

The Salisbury interchange, located roughly three miles east of the Edmonton city limits, is basically a cloverleaf incorporating outer connections of typical design and inner loops comprised of two parabolic tapers and three circular curves. This area is unique in that roughly 1500 feet east of the interchange, while travelling west, the highway diverges from two lanes (i.e. one lane in each direction) with a posted speed limit of 45 mph, to four lanes (i.e. two lanes in each direction) with a speed limit of 60 mph. West of the interchange the speed limit is 70 mph as the highway takes on freeway design standards with complete access control.

The study ramp was the outer connection which joined the Sherwood Park Freeway from the east and Highway Number 14X to the north. The ramp has a posted exit speed of 30 mph. This ramp was chosen primarily because of the number of vehicles which had crashed into lamp standards along the left hand side of the ramp. The ramp itself consists of a 13°49'50" circular curve turning to the right, a 12°00' circular curve turning to the left and finally a 14°00' circular curve turning to the right which was connected to Highway 14X by a parabolic taper of 164 feet. In addition, the ramp has a 567 foot deceleration lane preceded respectively by 20:1 and 40:1 tapers. The initial circular

curve was the one drivers were having the most difficulty negotiating. In its entirety the ramp consists of a 10 foot right shoulder, a 14 foot lane and a 6 foot left shoulder. There are no obstructions on the right hand side of the ramp; lamp poles, spaced at roughly 100 foot intervals, and a guardrail parallel the entire left pavement edge of the ramp. The complete interchange is shown in FIGURE A.1. A comparison between recommended design and the constructed ramp can be found in Chapter VII and APPENDIX E.

A.2 Bremner Interchange

The Bremner interchange, located approximately 8 miles east of the Edmonton city limits, is also a cloverleaf design incorporating outer connections of recommended design and inner loops with two parabolic tapers and three circular curves. The problem area at this location involved the inner loop connecting Highway Number 16 from the west to Highway Number 55 to the north. The posted speed limit on Highway Number 16 is 65 mph. The ramp has a posted exit speed of 25 mph. The study ramp consists of an initial parabolic taper of 150 feet, followed by three circular curves to the right of the radii 150 feet, 230 feet and 150 feet respectively, and, finally, another 150 parabolic taper. The ramp throughout consists of a 10 foot right shoulder, a 14 foot lane and a 6 foot left shoulder. There are no obstructions on the right hand side of the pavement; a guard-rail starting approximately 160 feet from the diverge point

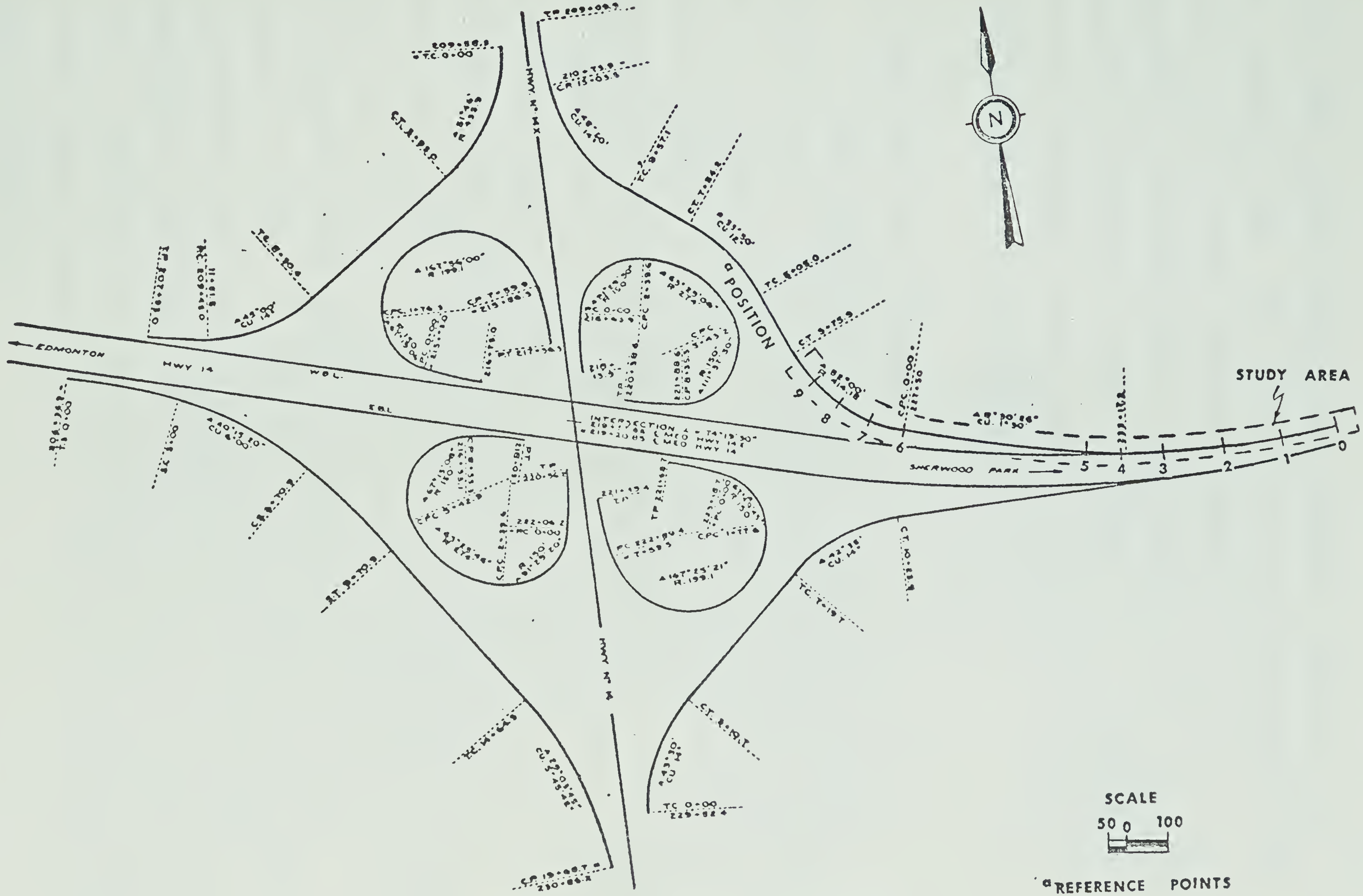

FIGURE A.1 SALISBURY INTERCHANGE

of the ramp and the through lane pavement, parallels the left pavement edge. The critical area of this ramp was the initial curve and the study was confined to vehicles approaching and travelling on this portion of the ramp. The complete interchange is shown in FIGURE A.2. A comparison between recommended design and the constructed ramp can be found in Chapter VII and APPENDIX E.

A.3 Groat Road and 107 Avenue

This interchange provided information on merging maneuvers only. It is a diamond-type interchange of standard design. The merging ramp, in the study area, varied from an initial width of 28 feet (i.e. position one) to a width of 22 feet (i.e. position four) at the merge point. Lamp poles are located on the right hand edge of the merge lane roughly three feet from the pavement edge; no obstructions are located between the merge ramp and the through lane. The posted speed limit on Groat Road in the merge area is 30 mph. Groat Road itself is situated in a ravine and because of this has numerous curves since it follows the path of a once meandering stream. A guardrail in the median of Groat Road separates the two lanes in each direction. The pavement downstream of the merge area funnels abruptly to form 11 foot lanes. Upstream of the merge area Groat Road consists of two 14 foot lanes in each direction. The entire interchange is shown in FIGURE A.3.

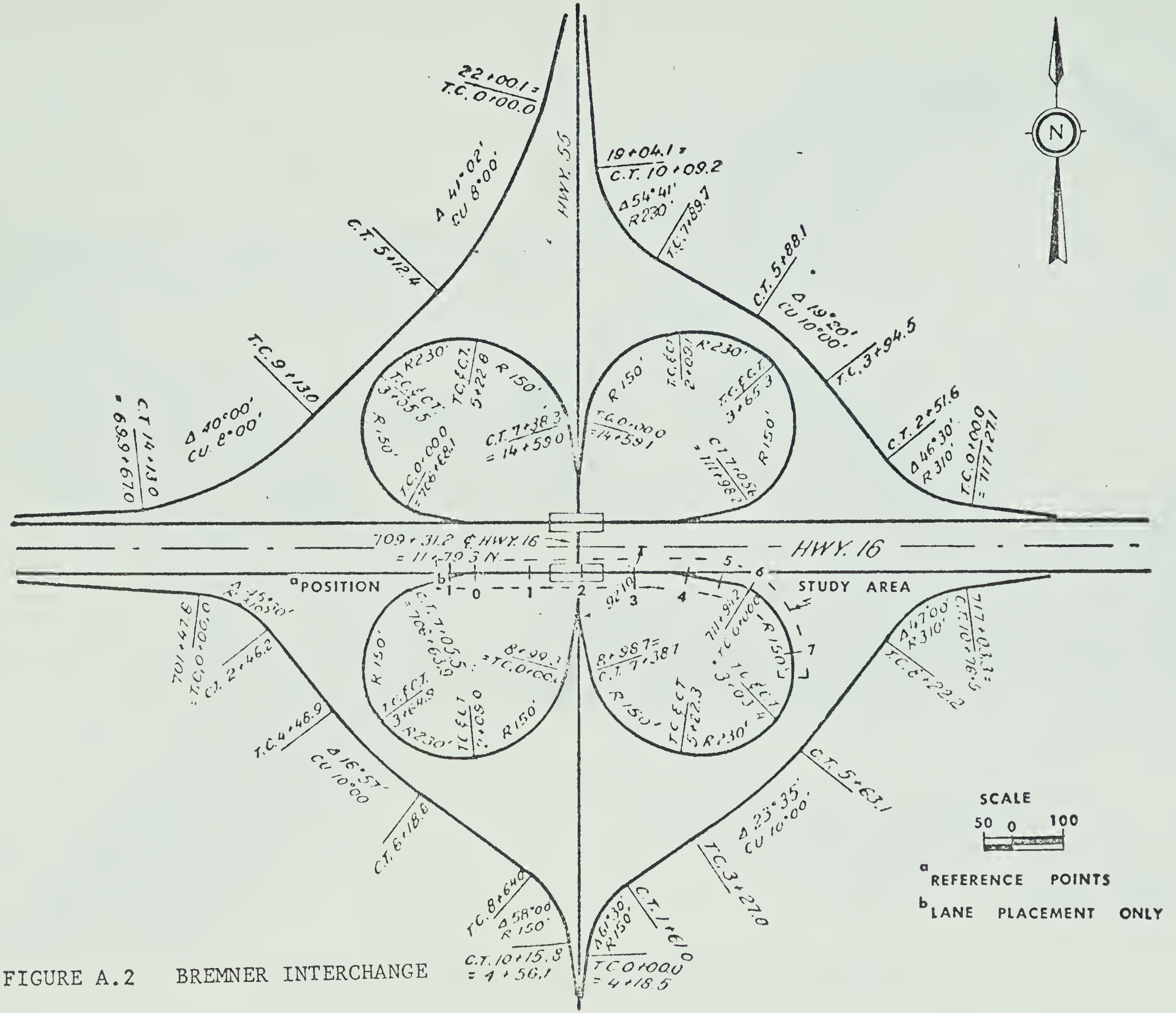

FIGURE A.2 BREMNER INTERCHANGE

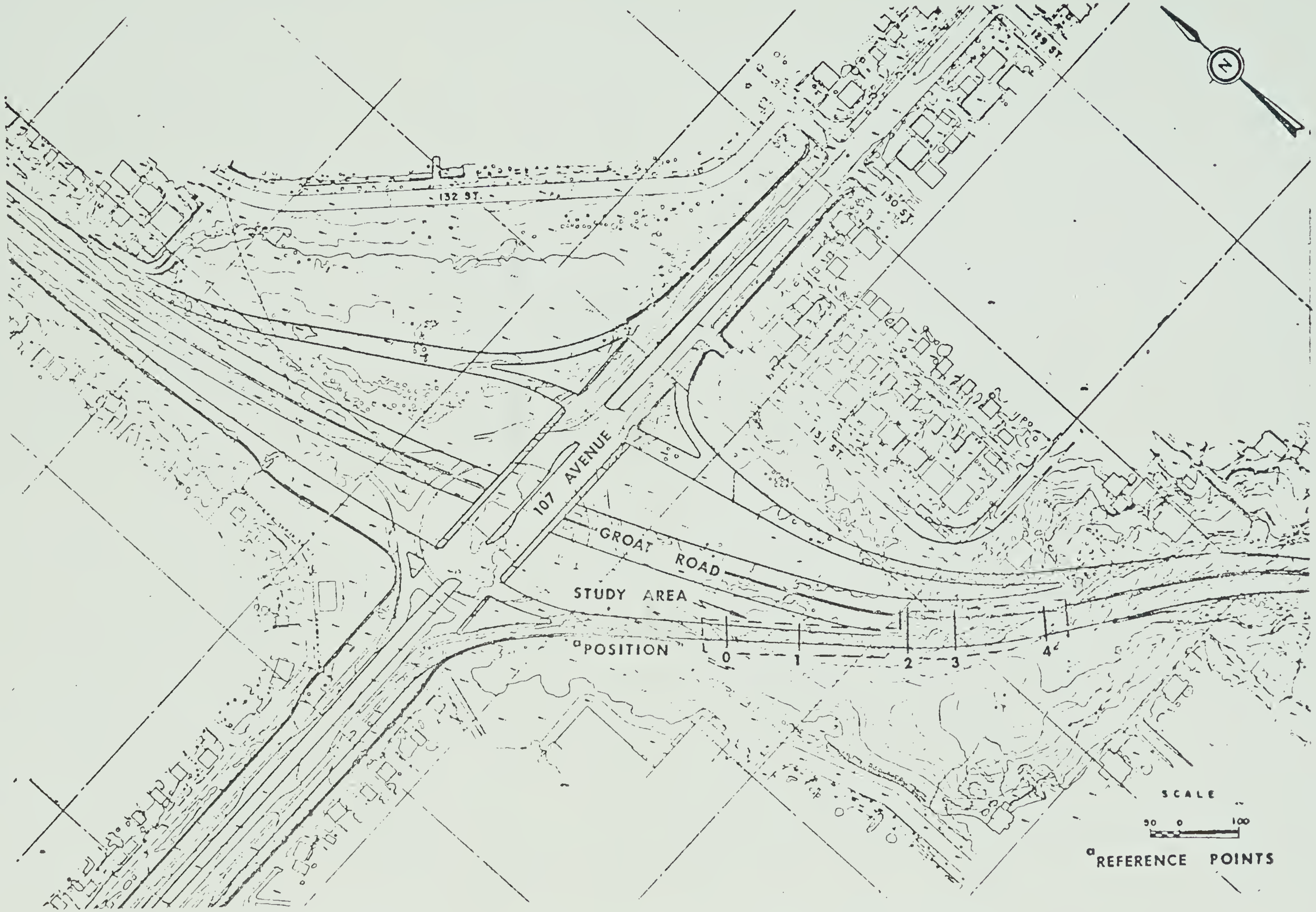

FIGURE A.3 GROAT ROAD AND 107 AVENUE INTERCHANGE

APPENDIX B

TEST EQUIPMENT

In Chapter V a brief description was given of the equipment used during the research. This appendix provides a more specific description of each of the individual items.

B.1 Camera and Accessories

B.1.1 Camera and Lens

The camera used was a 16 mm time-lapse "Paillard Bolex" which held 100 feet of film. The time-lapse feature of the camera incorporated a circuit breaker which automatically opened and closed the shutter and advanced the film one frame at whatever time was set on the cycle timer, e.g. one frame per second. This characteristic of the camera allowed the camera to be aimed at the required study area at the start of the days filming and, except for winding every 10 minutes, left untouched. The camera winding was necessary because of a spring within the camera. A frame counter and a film footage recorder furnished a check on the film progression during filming sessions.

A "P. Agenieux Retrofocus R21" French made lens with a focal length of 10 mm was used during all filming sessions. This was the largest lens available from the Motion Picture branch of the U. of A. and was used because its 54 degree horizontal angle field of view and 42 degree vertical angle field of view allowed the greatest area to be filmed without moving the camera. This lens was also especially useful since everything from three feet in front of the camera to infinity was in focus. This feature

permitted a clock placed three feet in front of the camera and vehicles up to 1500 feet away to be in focus. The correct lens setting was obtained every morning by using a standard light meter.

B.1.2 Cycle Timer

An intermittent current flow into the camera was obtained by using a repeat cycle timer. This instrument acted as a circuit breaker by automatically holding and releasing, at regular intervals, the current which entered the power supply regulator. Consequently, any frame interval could be chosen from 0-10 seconds per frame through the adjustment of a control knob. For the purposes of this study, since the peak hour was to be filmed, an interval of one frame per second was chosen because it would provide slightly over one hour of continuous filming with 100 feet of film, i.e.

$$1 \text{ frame height} = 8 \text{ mm}$$

$$100 \text{ feet of film} = 25.4 \ \frac{\text{mm}}{\text{in.}} \times 12 \ \frac{\text{in.}}{\text{ft}} \times 100 \text{ ft.} = 30480 \text{ mm}$$

$$\text{filming time @ 1 frame/sec}$$

$$= 30480 \text{ mm} \times \frac{1 \text{ frame}}{8 \text{ mm}} \times \frac{1 \text{ sec}}{1 \text{ frame}} \times \frac{1 \text{ min}}{60 \text{ sec}}$$

$$= \frac{30480}{480} = 63.5 \text{ minutes.}$$

This apparatus was connected to the power supply regulator and to the inverter.

B.1.3 Power Supply Regulator

The power supply regulator controlled the amount of current which intermittently entered the camera to operate the automatic shutter release. It had a voltage range of 0-40 volts and a current range of 0-500 milliamps, although 600 milliamps could be obtained. For the purpose of this research, 26 volts and 600 milliamps was used. These values were readily acquired by a manipulation of the range selector controls on the front of the apparatus. In addition, a combination current-voltmeter incorporated into the apparatus provided a check on the voltage and current input to the camera. This equipment was connected to the camera, cycle timer and the inverter.

B.1.4 Inverter and Battery

The necessity of a 110 volt AC power input to the camera required the use of a 12 volt car battery to supply the power and an inverter or converter to convert the energy of the battery to electricity. The inverter employed in this study had a 12 volt DC input and a 110 volt AC output. It had a range selector for power supply from low to high. Throughout all the filming sessions, it was set at low. Since it had a plug-in for only one two-prong plug, a socket was used which provided plug-ins for three electrical instruments instead of one. All three outlets were used:

1. The power supply regulator

2. The cycle timer
3. A clock.

The clock, with a sweep second hand, was superimposed into the field of view of the camera to furnish a check on the frame interval. The inverter, was connected to the battery by two spring-loaded clamp-type connectors.

B.1.5 Film

The film used in all filming sessions was 16 mm Kodak Plus-X Reversal film, Type 7276. Only one film taken was spoiled and whether this was the fault of the film or the photographer is difficult to determine. All other films provided adequate results.

B.2. Film Analysis Equipment

B.2.1. Projector

The projector employed was a special analyzer projector which permitted single frame viewing of the film. The projector was a "L W Photo-Optical Data Analyzer." It had a frame counter which was accurate to within plus or minus one frame. Its light source was a 750 watt DDB bulb and, because of an efficient blower, the film was kept from warping due to excessive lamp heat. Its reels handled easily the 100 foot film reels analyzed and could have handled 300 feet of film with no problem. Winding of the film was a manual operation and care had to be taken that sprocket holes

were firmly placed around sprockets on the winding wheels.

B.2.2 Remote Control Device

The single frame advancement feature of the projector was the basic analytical characteristic. Frames could be advanced one at a time by means of a remote control device which contained three major controls:

1. A button in the top left hand corner
2. A knob in the middle of the device
3. A knob in the bottom right hand corner.

By pushing the button in the top left hand corner, single frame advancement was possible. By selecting the frame speed with the knob in the bottom right hand corner and placing the middle knob on "auto", any one of several frame speeds (i.e. 1,2,4,6,8 or 10 frames/second) could be obtained automatically. In addition, the film could be viewed at either 16 or 24 frames per second by placing the center knob on "cine". Forward and reverse film viewing was possible for all speeds.

B.2.3 Screen and Mirror

The analysis equipment also incorporated a mirror and a translucent screen. The film image was initially projected onto the mirror and was reflected, in turn, onto the screen set into a desk. The screen was $1\frac{1}{2}$ by $2\frac{1}{2}$ feet and provided a sufficiently large picture by merely adjusting the position of the projector and focusing the lens.

APPENDIX C

COMPUTER PROGRAMS

C.1 Sorting Computer Program

The computer program shown in FIGURE C.1 serves several purposes. Firstly, it takes an array of numbers (i.e. K(I,J)) and sorts them in descending order of magnitude (i.e. Z(N,J)). It then determines the sum (i.e. SUMZ (I,J)) and the arithmetic mean (i.e. AR(J)). Following this, the standard deviation (i.e. SD(J)) is calculated. The percentile of each sorted number is then determined (i.e. PRC(N,J)).

The sorted array, the percentile, the arithmetic mean and the standard deviation form the output. The array and percentile is printed in 10 columns and up to 56 rows per page. The arithmetic mean and standard deviation are provided directly beneath the array.

The program is repetitive and, upon completion of one group of numbers, proceeds to the next. All output is presented on a new page.

The program is written for any general array. Only titles, dimension statements and format statements need be changed for different arrays.

The program shown in FIGURE C.1 is the one used to analyze the Salisbury truck velocity data.

```
C     SALISBURY TRUCK VELOCITY DATA
      INTEGER B,D,E,F,G,H,O,R,V,W,X,Z,PRC,A,AA
      DIMENSION K(40,9),Z(40,9),C(1),S(40,9),SUMZ(40,9),T(40,9),SUMT(40,
     19),PRC(40,9),AR(40),SD(40)
      DO 10 J=1,9
      READ (5,1000) (K(I,J),I=1,12)
 1000 FORMAT (12I3)
   10 CONTINUE
      DO 11 J=1,9
      A=12
      AA=A+1
      N=1
      L=1
    5 C(1)=0.0
      DO 20 I=1,A
      IF(K(I,J) .GE. C(1)) GO TO 15
      GO TO 20
   15 C(1)=K(I,J)
      L=I
   20 CONTINUE
      Z(N,J)=C(1)
      N=N+1
      K(L,J)=0
      IF(N .EQ. AA) GO TO 21
      GO TO 5
   21 SUMZ(1,J)=Z(1,J)
      DO 30 I=2,A
      Q=I-1
      SUMZ(I,J)=SUMZ(Q,J)+Z(I,J)
   30 CONTINUE
      AR(J)=SUMZ(A,J)/A
      DO 40 I=1,A
      S(I,J)=Z(I,J)-AR(J)
      T(I,J)=S(I,J)**2
   40 CONTINUE
      SUMT(1,J)=T(1,J)
      DO 50 I=2,A
      Q=I-1
      SUMT(I,J)=SUMT(Q,J)+T(I,J)
   50 CONTINUE
      SD(J)=SQRT(SUMT(A,J)/(A-1))
      DO 60 N=2,A
      M=N-1
      IF(N .GT. 2) GO TO 35
      PRC(M,J)=100-((100*M)/AA)
   35 IF(Z(N,J) .EQ. Z(M,J)) GO TO 25
      PRC(N,J)=100-((100*N)/AA)
      GO TO 60
   25 PRC(N,J)=PRC(M,J)
   60 CONTINUE
   11 CONTINUE
      J=1
      B=2
      D=3
      E=4
      F=5
      G=6
      H=7
      R=8
```

CONT.

FIGURE C.1 SORTING COMPUTER PROGRAM

```
      V=9
      WRITE (6,1500) J,B,D,E,F,G,H,R,V
 1500 FORMAT (1H1,20X,9HSALISBURY,5X,6HTRUCKS,9X,13HVELOCITY(MPH)//(4X,8
     1HPOSITION,I9,I13,I13,I13,I13,I13,I13,I13,I13)//(17X,9HSPEED - %,4X
     1,9HSPEED - %,4X,9HSPEED - %,4X,9HSPEED - %,4X,9HSPEED - %,4X,9HSPE
     1ED - %,4X,9HSPEED - %,4X,9HSPEED - %,4X,9HSPEED - %)//)
      DO 23 I=1,12
      WRITE (6,2700) Z(I,1),PRC(I,1),Z(I,2),PRC(I,2),Z(I,3),PRC(I,3),Z(I
     1,4),PRC(I,4),Z(I,5),PRC(I,5),Z(I,6),PRC(I,6),Z(I,7),PRC(I,7),Z(I,8
     1),PRC(I,8),Z(I,9),PRC(I,9)
 2700 FORMAT (19X,I2,2X,I3,6X,I2,2X,I3,6X,I2,2X,I3,6X,I2,2X,I3,6X,I2,2X,
     1I3,6X,I2,2X,I3,6X,I2,2X,I3,6X,I2,2X,I3,6X,I2,2X,I3)
   23 CONTINUE
      WRITE (6,3500) (AR(J),J=1,9),(SD(J),J=1,9)
 3500 FORMAT (///3X,17HARITHMETIC MEAN =,1F6.2,1F12.2,7F13.2//(1X,20HSTA
     1NDARD DEVIATION =,1F5.2,1F12.2,7F13.2))
      STOP
      END
```

FIGURE C.1 SORTING COMPUTER PROGRAM

C.2 Average Deceleration Rate Computer Program

The program shown in FIGURE C.2 solves for "a" in the formula:

$$v_o^2 - v_i^2 = 2as$$

where: v_o = final velocity (ft/sec)

v_i = initial velocity (ft/sec)

a = acceleration (ft/sec/sec) or deceleration, if negative

s = distance (ft) over which v_o and v_i occurred.

The average velocity values in mph (i.e. S(I)) and distances in feet (i.e. K(I)) are read in. The distances are multiplied by 2 (i.e. M(I)). The velocity in mph is changed to feet per sec (i.e. B(I)) and squared (i.e. C(I)). The value of the deceleration, D(I), or acceleration, is then computed.

Only titles, dimension and format statements need be changed to suit the particular output demands of the user.

The program shown in FIGURE C.2 was used to determine the average Salisbury truck deceleration rates.

```
      DIMENSION S(10),B(10),K(10),C(10),D(10),M(10)
      N=9
      READ (5,1000) (S(I),I=1,N)
 1000 FORMAT (10F7.2)
      L=N-1
      READ (5,1500) (K(I),I=1,L)
 1500 FORMAT (8I5)
      A=88.0/60.0
      DO 10 I=1,L
      M(I)=K(I)*2
   10 CONTINUE
      DO 25 I=1,N
      B(I)=A*S(I)
      C(I)=B(I)**2
   25 CONTINUE
      L=N-1
      DO 20 I=1,L
      J=I+1
      D(I)=(C(J)-C(I))/M(I)
   20 CONTINUE
      WRITE (6,2000)
 2000 FORMAT (1H120X,62HSALISBURY TRUCK DECELERATION RATES(FEET PER SECO
     1ND PER SECOND)//(10X,10HSPEED(MPH),10X,10HSPEED(FPS),10X,14HDISTAN
     1CE(FEET),10X,19HDECELERATION(FPSPS))/)
      DO 30 I=1,N
      K(N)=0
      D(N)=0.00
      WRITE (6,2500) S(I),B(I),K(I),D(I)
 2500 FORMAT (10X,1F7.2,13X,1F7.2,17X,I3,22X,1F6.2/)
   30 CONTINUE
      STOP
      END
```

FIGURE C.2 AVERAGE DECELERATION RATE COMPUTER PROGRAM

APPENDIX D

STATISTICAL ANALYSIS OF OBSERVED VELOCITIES

D.1 Salisbury Speeds

D.1.1 F Test

$$F = \frac{s_1^2}{s_2^2} \quad \text{where } s_1 > s_2$$

Cars: $n = 209$ Trucks: $n = 12$

$$v = n - 1$$

(See TABLE VII.1 for standard deviation values)

	Position	6	7	8	9
Standard Deviation s	Cars:	4.81	5.35	5.27	5.86
	Trucks:	4.34	3.92	4.00	5.14
	F =	$\frac{(4.81)^2}{(4.34)^2}$	$\frac{(5.35)^2}{(3.92)^2}$	$\frac{(5.27)^2}{(4.00)^2}$	$\frac{(5.86)^2}{(5.14)^2}$
	F =	1.23	1.85	1.74	1.30

From Neville and Kennedy (1964), Table A.10:

@ 5% level of significance with

$v_1 = \infty$, since 209-1 > 120;

and $v_2 = 11$

Tabulated F = 2.40

Since all computed values of F are less than the tabulated F value, the null hypothesis is accepted and there is no significant difference between the distributions being compared.

D.1.2 t test

$$t = \frac{\bar{x}_1 - \bar{x}_2}{s_d}$$

$$s_c^2 = \frac{s_1^2(n_1-1) + s_2^2(n_2-1)}{(n_1-1) + (n_2-1)}$$

$$s_d = s_c\sqrt{\frac{n_1+n_2}{n_1 n_2}} \; ; \quad \sqrt{\frac{n_1+n_2}{n_1 n_2}} = \sqrt{\frac{221}{2510}} = 0.297$$

(See TABLE VII.1 for average velocity values)

Position 6

Average Velocity $\bar{x}$

Cars: 37.11

Trucks: 33.83

$$s_c^2 = \frac{(4.81)^2\ (208) + (4.34)^2\ (11)}{208 + 11}$$

$$s_c^2 = 31.4$$

$$s_c = 5.60$$

$$s_d = 1.66$$

$$t = \frac{37.11 - 33.83}{1.66} = 1.98$$

Position 7

Average Velocity $\bar{x}$

Cars: 34.59

Trucks: 33.50

$$s_c^2 = \frac{(5.35)^2\ (208) + (3.92)^2\ (11)}{208 + 11}$$

$$s_c^2 = 35.0$$

$$s_c = 5.92$$

$$s_d = 1.76$$

$$t = \frac{34.59 - 33.50}{1.76} = 0.62$$

Position 8

Average Velocity $\bar{x}$

Cars: 37.11

Trucks: 36.17

$$s_c^2 = \frac{(5.27)^2 (208) + (4.00)^2 (11)}{208 + 11}$$

$$s_c^2 = 34.4$$

$$s_c = 5.86$$

$$s_d = 1.74$$

$$t = \frac{37.11 - 36.17}{1.74} = 0.54$$

Position 9

Average Velocity $\bar{x}$

Cars: 40.52

Trucks: 39.58

$$s_c^2 = \frac{(5.86)^2 (208) + (5.14)^2 (11)}{208 + 11}$$

$$s_c^2 = 45.9$$

$$s_c = 6.77$$

$$s_d = 2.01$$

$$t = \frac{40.52 - 39.58}{2.01} = 0.47$$

From Neville and Kennedy (1964), Table A.8:

@ a 5% level of significance and the number of

degrees of freedom $= n_1 + n_2 - 2 = V$

With $V = \infty$ since $209 + 12 - 2 > 120$

Tabulated $t = 1.960$

Therefore, only at the ramp entrance (i.e. Position 6), where the computed t value is greater than the tabulated t value, do car velocities differ significantly from truck velocities.

D.2 Bremner Speeds

D.2.1 F test

$$F = \frac{s_1^2}{s_2^2} \quad \text{where } s_1 > s_2$$

Cars: $n = 500$ Trucks: $n = 40$

$$v = n-1$$

(see TABLE VII.3 for standard deviation values)

	Position	4	5	6	7
Standard Deication s	Cars:	5.11	3.47	3.21	4.03
	Trucks:	4.30	3.07	3.25	3.56
	F =	$\frac{(5.11)^2}{(4.30)^2}$	$\frac{(3.47)^2}{(3.07)^2}$	$\frac{(3.25)^2}{(3.21)^2}$	$\frac{(4.03)^2}{(3.56)^2}$
	F =	1.42	1.28	1.02	1.28

From Neville and Kennedy (1964), Table A.10:

@ a 5% level of significance and with

$v_1 = \infty$, since $500 - 1 > 120$

$v_2 = 39$

Tabulated F = 1.52 for all positions except 6

where: $v_1 = 39$

$v_2 = \infty$ since s_1 must be $> s_2$

and the Tabulated F = 1.40 for position 6.

Since all computed F vaules are less than the tabulated F value, the null hypothesis is accepted and there is no significant difference between the distributions being compared.

D.2.2 t test

$$t = \frac{\bar{x}_1 - \bar{x}_2}{s_d}$$

$$s_c^2 = \frac{s_1^2 (n_1-1) + s_2^2 (n_2-1)}{(n_1-1) + (n_2-1)}$$

$$s_d = s_c \sqrt{\frac{n_1+n_2}{n_1 n_2}} \; ; \; \sqrt{\frac{n_1+n_2}{n_1 n_2}} = \sqrt{\frac{540}{20000}} = 0.164$$

(see TABLE VII.3 for average velocity values)

	Position	4
Average Velocity $\bar{x}$	Cars:	33.97
	Trucks:	33.22

$$s_c^2 = \frac{(5.11)^2 (499) + (4.30)^2 (39)}{499 + 39}$$

$$s_c^2 = 25.6$$

$$s_c = 5.06$$

$$s_d = 0.83$$

$$t = \frac{33.79 - 33.22}{0.83} = 0.69$$

	Position	5
Average Velocity $\bar{x}$	Cars:	25.42
	Trucks:	27.35

$$s_c^2 = \frac{(3.47)^2 (499) + (3.07)^2 (39)}{499 + 39}$$

$$s_c^2 = 11.95$$

$$s_c \quad 3.45$$

$$s_d = 0.57$$

$$t = \frac{27.35 - 25.42}{0.57} = 3.39$$

Position 6

Average Velocity $\bar{x}$

Cars: 21.32

Trucks: 21.80

$$s_c^2 = \frac{(3.21)^2 (499) + (3.25)^2 (39)}{499 + 39}$$

$$s_c^2 = 10.4$$

$$s_c = 3.23$$

$$s_d = 0.53$$

$$t = \frac{21.80 - 21.32}{0.53} = 0.91$$

Position 7

Average Velocity $\bar{x}$

Cars: 24.19

Trucks: 20.07

$$s_c^2 = \frac{(4.03)^2 (499) + (3.56)^2 (39)}{499 + 39}$$

$$s_c^2 = 16.0$$

$$s_c = 4.00$$

$$s_d = 0.66$$

$$t = \frac{24.09 - 20.07}{0.66} = 6.10$$

From Neville and Kennedy (1964), Table A.8:

@ a 5% level of significance and the number of degrees of freedom $= n_1 = n_2 - 2 = V$

with $V = \infty$ since $500 + 40 - 2 > 120$

Tabulated $t = 1.96$

Therefore, since the computed values of t at positions 5 and 7 are greater than the tabulated t value, there is a significant difference between car and truck velocities at these positions. Positions 4 and 6 show no significant difference, since the computed values of t are less than the tabulated t value.

D.3 Groat Road and 107 Avenue

D.3.1 F test

$$F = \frac{s_1^2}{s_2^2} \quad \text{where } s_1 > s_2$$

Cars: $n = 695$ Trucks: $n = 35$

$$v = n - 1$$

(see TABLE VII.5 for standard deviation values)

	Position	1	2	3	4
Standard Deviation s	Cars:	3.99	8.60	5.92	3.94
	Trucks:	2.27	2.71	3.30	3.16
	F =	$\frac{(3.99)^2}{(2.27)^2}$	$\frac{(8.60)^2}{(2.71)^2}$	$\frac{(5.92)^2}{(3.30)^2}$	$\frac{(3.94)^2}{(3.16)^2}$
	F =	3.10	10.30	3.10	1.56

From Neville and Kennedy (1964), Table A.10:

@ a 5% level of significance and with

$v_1 = \infty$, since $695 - 1 > 120$

$v_2 = 34$

Tabulated $F = 1.58$

Since the computed values of F for positions 1 - 3

are greater than the tabulated F value, the null hypothesis is rejected and it must be concluded that there is a significant difference between the variances being compared. The F value of position 4 is almost equal to the computed F value and is on the border between being significantly different or not. Since the t test assumes that the two variances must not be significantly different and since it has been shown that there is a significant difference between the variances of this sample, there is no point in performing the t test.

APPENDIX E

RURAL STUDY RAMP GEOMETRICS

E.1 General

In the comparison of recommended design practice with "as constructed" geometrics, one very important consideration is worthy of note. From the Annual Reports of the Department of Highways and Transport of the Province of Alberta for the years 1963-1969, it was shown that the Bremner interchange was designed in the period 1963-64. This indicated the interchange was designed before the most recent AASHO publication of recommended rural highway design in 1965. There were 28 rural interchanges constructed in Alberta by 1965. The contract for the Salisbury interchange was awarded in 1964, indicating a design which was also pre-AASHO (1965). The Salisbury interchange was completed in the 1965-66 construction season, with a total of 31 rural interchanges completed by this time in Alberta.

The design of these interchanges would therefore, be largely based on the rural highway design recommendations published by AASHO in 1954 (i.e. the previous design manual) and by CGRA before 1965. However, between the period 1954-1965, numerous improvements were made in automobiles, resulting in a general increase in automobile performance. In order for highway design to keep pace with improved automobile design, more up-to-date highway design recommendations were published by AASHO in 1965. It is on the basis of these 1965 recommendations that a comparison with the geometrics of the Salisbury and Bremner inter-

changes is made. Based on the most recent design proposals, several design features of the Salisbury and Bremner interchanges fall at and below current recommended minimum practices (e.g. exit speeds of 30 mph and 25 mph, respectively).

As of March, 1969, there were 53 rural interchanges in Alberta, with 31 of these constructed by 1966. This indicates that at least 31 interchanges were designed before 1965 and probably now incorporate design features at and below the most recent minimum design recommendations (i.e. those of AASHO (1965)). The Salisbury and Bremner interchanges provide an example of those interchanges designed before 1965.

TABLE E.1A
SALISBURY STUDY RAMP GEOMETRICS

DESIGN CRITERIA	[a] RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
HIGHWAY DESIGN SPEED	60 - 70 MPH	70 MPH	O. K.
RAMP DESIGN SPEED	DESIRABLE 45 - 50 MPH MINIMUM 30 MPH	$R = V^2/15(e + f)$ $V = \sqrt{15R(e + f)}$ $V = 36.95$ MPH	THE HIGHER THE DESIGN EXIT SPEED THE BETTER; 37 MPH ON THE LOW SIDE
DEGREE of CURVE	MAX. 21° AT 30 MPH MAX. 11.3° AT 40 MPH	D = 13° 49' 56" R = 414.18 FT	O.K. FOR 30 MPH EXIT SPEED; MORE CRITICAL AS EXIT VELOCITY INCRS.
e	0.02 - 0.08 FT/FT	0.06 FT/FT	O. K.
f	0.11 - 0.16	0.16	SPEEDS GTR. THAN 35 UNSAFE; f SMALL
PAVEMENT WIDTH	15 FT	30 FT	O. K.
LANE WIDTH	12 FT	14 FT	O. K.

[a] RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7

TABLE E.1B
SALISBURY STUDY RAMP GEOMETRICS

DESIGN CRITERIA	[a]RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
SHOULDERS	MAX. 10 - 12 FT MAX. 4 - 6 FT	RIGHT SHOULDER = 10 FT LEFT SHOULDER = 6 FT	O. K.
LATERAL CLEARANCE	DESIRABLE = 6 FT MINIMUM = 3 FT	RT. SHLDR. = NO OBSTRUCTIONS LT. SHLDR. = 6 FT	O. K.
SPIRAL CURVE	AT 30 MPH, L = 110 FT MIN AT 40 MPH, L = 130 FT	NONE	PROVIDES MORE NATURAL DRIVING PATH ON CURVE
DECEL. LENGTH	AT 30 MPH, L = 525 FT MIN AT 40 MPH, L = 425 FT (Both @ HWY SPEED 70 MPH)	L = 567 FT	O. K.
STOPPING SIGHT DISTANCE	AT 30 MPH, L_{min} = 200 FT AT 40 MPH, L_{min} = 275 FT	SSD = $1.47PV+V^2/30(f \pm g)$ SSD = 260 FT @ V = 30 MPH	INADEQUATE FOR EXIT SPEEDS GTR. THAN 35 MPH

[a]RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7

TABLE E.1C
SALISBURY STUDY RAMP GEOMETRICS

DESIGN CRITERIA	[a]RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
GRADE	AT 30 MPH, 6 - 8% MAX.	+ 0.4%	O. K.
SUPER. RUN—OFF RATE	AT DESIGN SPEED 30 MPH PER STATION = 0.06 FT/FT PER 25 FT = 0.015 FT/FT	PER STATION = 0.06 FT/FT	O. K.
CIRCULAR ARC LENGTH	AT 400 FT RADIUS: DESIRABLE = 180 FT MINIMUM = 120 FT	AT R = 414 FT ARC = 300 FT	O. K.
TERMINAL DESIGN	NOSE OFFSET = 4 - 12 FT LENGTH OF NOSE TAPER = 15 FT MIN.	NOSE OFFSET = 4 FT LENGTH OF NOSE TAPER = 70 FT	O. K. FULL WIDTH STABILIZED SHOULDER PROVIDES NOSE OFFSET AND ACCOMMODATES CORRECTIVE MANEUVERS

[a] RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7

TABLE E.2A
BREMNER STUDY RAMP GEOMETRICS

DESIGN CRITERIA	°RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
HIGHWAY DESIGN SPEED	60 - 70 MPH	70 MPH	O. K.
RAMP DESIGN SPEED	DESIRABLE = 45 - 50 MPH MINIMUM = 30 MPH	$R = V^2/15(e+f)$ $V = \sqrt{R\ 15\ (e+f)}$ $V = 22.25$ MPH	INADEQUATE, TOO GREAT A SPEED CHANGE REQ'D FROM THROUGH LANES
DEGREE of CURVE	MAX. 21° at 30 MPH MAX. 11.3° AT 40 MPH	D = 38.2° R = 150 FT	INADEQUATE FOR RURAL HWY. DESIGN; BETTER SUITED TO URBAN AREA
e	0.02 - 0.08 FT/FT	0.06 FT/FT	O. K.
f	0.11 - 0.16	0.16	SPEEDS GTR. THAN 25 MPH UNSAFE, f SMALL
PAVEMENT WIDTH	15 FT	30 FT	O. K.
LANE WIDTH	10 - 12 FT	14 FT	O. K.

°RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7

TABLE E:2B

BREMNER STUDY RAMP GEOMETRICS

DESIGN CRITERIA	[a]RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
SHOULDERS	MAX. = 10 - 12 FT MIN. = 4 - 6 FT	RIGHT SHOULDER = 10 FT LEFT SHOULDER = 6 FT	O. K.
LATERAL CLEARANCE	DESIRABLE = 6 FT MINIMUM = 3 FT	RT. SHLDR. = NO OBSTRUCTIONS LT. SHLDR. = 6 FT	O. K.
SPIRAL CURVE	AT 30 MPH, L_s = 110 FT MIN AT 40 MPH, L_s = 130 FT	150 FT	O. K.
DECEL. LENGTH	AT 25 MPH, L_{min} = 550 FT AT 30 MPH, L_{min} = 525 FT (BOTH AT HWY SPEED 70 MPH)	AUXILIARY LANE = 230 FT SPIRAL = 150 FT TOTAL LENGTH = 380 FT	INSUFFICIENT LENGTH AVAILABLE TO PERMIT COMFORTABLE DECELER. TO EXIT SPEED OF 25 MPH
STOPPING SIGHT DISTANCE	AT 30 MPH, SSD_{min} = 176 FT AT 40 MPH, SSD_{min} = 263 FT	$SSD = 1.47PV + V^2/30(f \pm g)$ SSD = 117 FT AT V= 25MPH	INADEQUATE FOR EXIT SPEEDS GTR. THAN 25 MPH

[a]RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7

TABLE E.2C

BREMNER STUDY RAMP GEOMETRICS

DESIGN CRITERIA	[a]RECOMMENDED DESIGN	AS CONSTRUCTED	COMMENTS
GRADE	AT 30 MPH, MAX. = 6 -8% AT 40 MPH, MAX. = 5 -7%	+ 0.667%	O. K.
SUPER. RUN — OFF RATE	AT DESIGN SPEED 30 MPH: PER STATION = 0.06 FT/FT PER 25 FT = 0.015 FT/FT	PER STATION = 0.06 FT/FT	O. K.
CIRCULAR ARC LENGTH	AT 150 FT RADIUS: DESIRABLE = 70 FT MINIMUM = 50 FT	AT R = 150 FT ARC = 230 FT	O. K.
TERMINAL DESIGN	NOSE OFFSET = 4 - 12 FT LENGTH OF NOSE TAPER = 15 FT MIN.	NOSE OFFSET = 5 FT LENGTH OF NOSE TAPER = 70 FT	O. K. FULL WIDTH STABILIZED SHOULDER PROVIDES NOSE OFFSET AND ACCOMMODATES CORRECTIVE MANEUVERS

[a]RECOMMENDATIONS TAKEN FROM "A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965" BY AASHO, CHAPTERS 3,7